The paper was published in the proceedings of the 17th African Regional Conference on Soil Mechanics and Geotechnical Engineering and was edited by Prof. Sw Jacobsz. The conference was held in Cape Town, South Africa, on October 07-09 2019.

Flood damage repair to Mambedi lower dam spillway

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ABSTRACT: Construction of embankment dams to collect and store water has for millennia been a means of augmenting water resources for farmers. This stored water, mainly gathered from rainfall, is then used for irrigation purposes of crops and it is also used as a source of drinking water for livestock. In February 2000 such an embankment dam, owned by Eastern Produce Estates SA, was damaged by an extreme rainfall event, causing the spillway of the dam to be washed away. This reduced the dam's capacity to store water, as water would discharge through the 100m wide section which was removed by the flood. The rainfall event of February 2000, almost reaching the regional maximum flood, caused the existing concrete overflow wall, to be undermined which ultimately lead to the failure of the dam through the spillway. Because of this failure, a donga of 100m wide and 15m deep was created from subsequent water flowing through this section, as the soil downstream of the earthen dam had an extremely dispersive nature. This paper will illustrate the remedial actions taken to reinstate the dam to its full capacity with the use of traditional embankment dam construction methods combined with the use of gabions, Mattresses and MSE walls for constructing a downstream spillway.

1 INTRODUCTION

The flow required for designing a weir structure will normally be the flow for a 1:50 year return period. A structure is seldom designed, due to economical constraints, for a 1:100 year or maximum flood, unless the structure is critical and could cause loss of life or the client has requested it. In the case of Mambedi lower dam, the existing concrete overflow structure had been undermined by a flood which almost reached the regional maximum flood in the year 2000, ultimately leading to the failure of the dam through the spillway.

The client (Eastern Produce Estates SA) requested that the breach in the dam wall be repaired as they want to utilize their existing lawful water right. PG Consulting Engineers were subsequently appointed by the client to reinstate the damaged dam wall and were asked to allow in the design to accommodate the repair to be implemented in 2 phases. The spillway discharge capacity must comply with dam safety standards as prescribed by SANCOLD which require the RMF to pass with no substantial damage.

PG Consulting Engineers used traditional embankment dam design methods to repair the breached section of the dam wall. For the spillway, they however approached Maccaferri Africa to assist in the channelization of the water, flowing from the crest of the dam overflow, when it has reached full supply level.

2 EMBANKMENT AND SPILLWAY DESIGN APPROACH

During the floods of February 2000 "mother nature" created an extensive "donga" through the spillway with approximate dimensions of 60m width and 20m depth due to no rock foundation and highly erodible soils. The challenge was to provide a suitable spillway wall and spillway channel with adequate protection to accommodate the RMF as prescribed by the SANCOLD Guidelines.

A combination of gabion baskets with reno mattresses have proven itself in similar conditions, so it was decided to approach Maccaferri to assist with the design.

For the repair of the damaged spillway, the existing embankment wall, which was still intact, had to be re-aligned to join the new spillway wall.

A new embankment wall had to be provided on the right bank, as an extension to the main wall. It was also decided to construct the new embankment walls with the same crest level as the original wall, thus incorporating Phase 2 as well.

In addition to the above, cut-off zones had to be provided and constructed to lengthen the seepage path during full supply state. These cut-off zones had to be taken to a depth of approximately 4-5m to an impermeable strata. The upstream slopes were designed as 1 in 3 and downstream slopes as 1 in 2, for normal

dam engineering standards. The spillway wall located between the new embankments also had to be provided with a cut-off zone with a depth of approximately 6m, to an impermeable medium. The slopes were designed to suit Maccaferri's requirements.

Material for the construction of the new embankments, cut-off zones and spillway wall, were sourced from within the dam basin. This represents the same material which was used to construct the existing main embankment. Material from the eroded spillway channel could mainly not be used due to the dispersive nature and had to be spoiled. This caused an additional cost on the project, not budgeted for. Sand for backfilling behind the Terramesh walls had to be imported from the nearby river, at additional expense.

Figure 1. View of donga at old spillway section

Figure 2. Upstream view of donga

Figure 3. General arrangement of new spillway structure.

3 DOWNSTREAM PROTECTION

Protecting river banks of natural or altered streams has historically always been a field concerning the engineering profession. This is done by interpreting statistical data in order to provide safe and reliable design solutions at peak floods (Di Pietro, 2000).

The techniques used to protect streambanks from erosion range from simple vegetation to massive retaining structures such as concrete, gabions or mechanically stabilized earth wall systems. There are however several factors influencing the choice of intervention technique which are directly dependent on the geotechnical, hydraulic and environmental properties (Di Pietro, 2000).

Slope stability requires the evaluation of the hydraulic and the soil geotechnical parameters while experience indicates that there are three main causes of instability:

- Instability due to progressive surface erosion (water run-off);
- Instability due to soil veneer sliding;
- Instability due to deep sliding failure (global instability).

In the case of Mambedi lower dam, the downstream donga had been formed by progressive erosion due to constant water run-off on the banks as well as the stream bed amplified by the dispersive nature of the insitu soil. The chosen downstream solution at Mambedi lower dam required the ability to provide adequate shear resistance to reduce rainfall impact, runoff and scour together with a formalization of the banks using a retaining structure. Thus, the chosen solutions required the provision of short term as well as long term stability.

The protection explored and ultimately chosen at Mambedi lower dam included the use of gabions, Reno mattresses and Terramesh systems. The various functions of each are explained in the subsequent sections.

4 STEPPED GABION WEIR

Stepped spillways and weirs are designed to enhance the rate off energy dissipation and has been used as such for over 3500 years (Chanson 2000). Stepped spillway flows are characterized by strong flow aeration which enhance the rate of energy dissipation and therefore reduces the size and cost of the downstream stilling structure. Moreover, consideration of the type of stepped spillway also needs to be considered. Studies conducted by Wuthrich & Chanson (2014) compared the effectiveness of energy dissipation for different step types in both nappe and skimming flow conditions. It was concluded that gabions dissipate energy more efficiently in nappe flow conditions because of the significant proportion of seepage flow while flat impervious stepped weirs handle skimming flow conditions more effectively due to the aeration efficiency. There is also a third alternative which they considered: gabions steps with a concrete capping. It was with this in mind that the design of a stepped gabion weir, with concrete capping, was considered as the solution downstream of the Mambedi lower dam spillway.

Maccaferri's MACRA 2 design software was used to model the weir structure at Mambedi lower dam to conduct a hydraulic and static analyses using Maccaferri products. The software uses first principles to model and ultimately design a satisfactory structure with the provided input parameters.

With the below input values, provided by PG Consulting engineers, the model yielded the following results as displayed in Figure 4:

- Regional maximum flood = 854 m^3/s
- River bed gradient = 1 %
- Channel width = 66 m
- Bank top elevation = 18 m
- Soil unit weight = 15 kN/m^3
- Soil Friction angle = 23 °

Run n.1

Design discharge	Q [m3/s]	854.00	Soil unit weight	[kN/m3]	15.00
River bed gradient	i [%]	1.00	Soil friction angle	(deg)	23.00
Roughness coefficient	n	0.033	Soil type		Sa
Soil Granulometry in the pool	dl [mm]	250.00	Gabion porosity	n	0.30
Channel Width	L3 [m]	66.00	Soil-weir friction angle	[deg]	10.00
Bank top elevation	fp [m]	18.00	Soil-foundation friction coefficient	f	0.70
Slope of left bank	Pl [deg]	90.00	Underpressure influence	Sol [%]	50.00
Slope of right bank	Pr [deg]	90.00	Bligh coefficient	Cb	4.00

Weir Data - Stepped Weir

Crest elevation	fg [m]	11.00
Crest width	Lg [m]	66.00
Slope of crest wings	Pg [deg]	90.00
Total weir height	H [m]	12.00
H/b		1/2

Summary of Hydraulic Results

Water elevation on crest	zg [m]	13.57
Water elevation downstream	z3 [m]	2.46
Backwater elevation	z0 [m]	14.86
Scour depth downstream	fb [m]	0.00
Water elevation at the nappe toe	z1 [m]	0.76
Tailwater depth sequent to z1	z2 [m]	6.34
Min. required basin length	LBas [m]	38.54
Min. required counterweir elevation	fc [m]	1.50

Figure 4. Results from MACRA 2 software.

The Software also provides the designer with a long section view as shown in Figure 5.

The results included a freeboard requirement of 2.46 m downstream in the channel after the required 38.54 m basin with a 1.5 m counter weir. Bank protection would thus be required up to the minimum required heights as illustrated in the above long section view as provided by the MACRA 2 software.

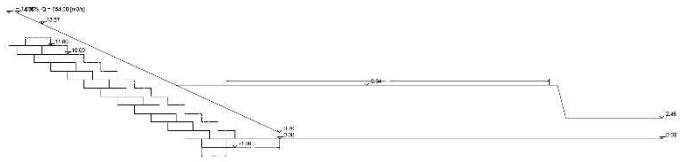


Figure 5. Long section view from MACRA 2.

5 BANK PROTECTION

Reinforcing a soil mass by including a material that is strong in tensile resistance is comparable to the behaviour of reinforced concrete. The mechanism of reinforcement between reinforced concrete and reinforced soil is however quite different. In reinforced soil, the soil-geosynthetic interface friction is where the bonding between soil and the reinforcement element is derived from. The reinforcement restrains lateral deformation of the soil next to the reinforcement element, through the interface friction, thereby increasing the stiffness and strength of the soil mass. The soil straining under loading will strain the geosynthetic through interface friction which in turn develops strength in the geosynthetic, thereby restraining the soil laterally.

Geosynthetic-reinforced soil systems can be adapted to a variety of site conditions, such as weak foundations or low-quality fill with presence of silt or clay and have successfully been used in critical structures such as retaining walls, embankments, shallow foundations and bridge abutments.

Maccaferri has extensive experience in designing and providing reinforcement materials for mechanically stabilized earth structures. For very tall MSE structures, one of Maccaferri's systems combines Terramesh units with geogrids to provide the suitable stability required for these types of structures. The geogrids in this system are considered to act as the primary reinforcement while the Terramesh units act as a secondary reinforcement, mainly considered as "facing units". In these structures the primary reinforcements' tensile strength is activated to ensure a sound structure in terms of internal and global stability with the desired safety factors.

Terramesh units, which are produced with a "tail" of double twisted wire mesh as a secondary reinforcement, provide local stability at the face, ensuring that no local mechanism of direct sliding, pull-out or rotational failure can occur. Very tall structures have been built all over the world with this solution and as such it was deemed to be a suitable solution for the stabilization of the banks while conducting a second function of erosion control downstream of the weir at Mambedi lower dam.

The design of the Terramesh walls at Mambedi lower dam were done using Maccaferri's specialized in-house design software based on the BS 8006:2012 (code of practice for Strengthened/reinforced soils and other fills) named Macstars 4.0.

The stability analyses for all the walls were done using the Limit Equilibrium Method based on the material properties provided by PG Consulting Engineers as given below.

- In situ Material:
  - Internal Friction Angle = 26°
  - Unit Weight = 13.37 kN/m^3
  - Cohesion = 0 kPa
- Structural Fill:
  - Internal Friction Angle = 33°
  - Unit Weight = 17.08 kN/m^3
  - Cohesion = 0 kPa
- Foundation Material:
  - Internal Friction Angle = 30°
  - Unit Weight = 15.00 kN/m^3
  - Cohesion = 0 kPa
- Clay Material:
  - Internal Friction Angle = 21°
  - Unit Weight = 16.32 kN/m^3 (17 kN/m^3 used)
  - Cohesion = 62 kPa (zero kPa used)

With the above input values, given geometry and requirements from MACRA 2, the Terramesh walls were designed according to BS 8006:2012 to produce safe designs. Checks carried out using Macstars 4.0 included internal stability checks, global stability checks, sliding checks, overturning checks and bearing checks. Using the Macstars 4.0 software, 3 sections of up to 15m high were designed in rapid draw down conditions, which provided satisfactory results in terms of safety and cost.

In addition to the standard designs, a subsoil drain was constructed behind the reinforced fill to maintain dry conditions within the fill. To limit the ingress of water into the fill from the channel side (face of the Terramesh wall), a geomembrane liner was installed directly behind the stone facing of the Terramesh walls as indicated on the typical section a-a in Figure 6.

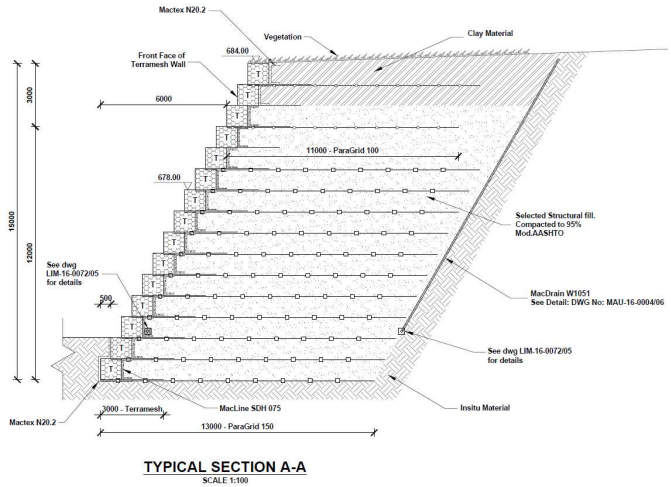


Figure 6. Typical section A-A.

6 DOWNSTREAM SCOUR PROTECTION

Erosion follows the principles that instability starts when velocities or shear stresses generate soil particles to move with the flow. The hydraulic stability is therefore measured when a critical velocity into the river is reached to start the initial motion of particles. This velocity will be different from soil to soil, and it will depend on the soil type, the depth of water and the geometry of the channel.

Another criterion of a more general nature is to consider the effective shear stress exerted on the surface and compare it with the allowable shear resistance typical of each type of material. Normally the criterion of the allowable velocities is more suitable to regular river sections (straight and trapezoidal in shape), as opposed to the criterion of the shear stresses, where the application is more realistic and representative of the effect of the water.

Table 1 (Fortier & Scobey 1926) gives the indicative water velocities generating instability in typical soils. We can see that when velocities exceed 1.5-2 m/s most materials available in nature start being eroded.

Table 1. Max allowable velocities for straight low gradient channels.

Material	Clear water V (m/s)	Water carrying colloidal silt V (m/s)
Fine sand	0.45	0.76
Sandy loam, non-colloidal	0.53	0.76
Silt loam, non-colloidal	0.6	0.91
Alluvial silts, non-colloidal	0.6	1.06
Ordinary firm loan	0.76	1.06
Volcanic ash	0.76	1.06
Stiff clay & alluvial silts, colloidal	1.14	1.52
Shales and hardpans	1.82	1.82
Fine gravel	0.76	1.52
Graded loam to cobbles non-colloidal	1.14	1.52
Graded silts to cobbles when colloidal	1.22	1.67
Coarse gravel, non-colloidal	1.22	1.82
Cobbles and shingles	1.52	1.67

Given that the expected velocities (up to 5.89 m/s) during a regional maximum flood event in the Mambedi lower dam channel will be higher than the allowable velocities for most soils, and that the soils at Mambedi lower dam displayed highly erodible behaviour, a complete lined channel was proposed and modelled in order to protect against scour downstream of the weir structure. To calculate the shear stress and velocities acting on the base of the channel, a section was modelled using Maccaferri's MACRA 1 software.

Full scale tests conducted on Maccaferri's range of erosion control systems yielded the allowable velocities and admissible shear stresses given in Table 2 and Table 3. Seeing that the allowable velocity of a 300mm Reno Mattress exceeded the expected velocities at Mambedi lower dam, the channel was modelled in MACRA 1 to confirm if the allowable velocity and admissible shear stresses meet the criteria of the system.

Table 2. Indicative values for tested erosion systems

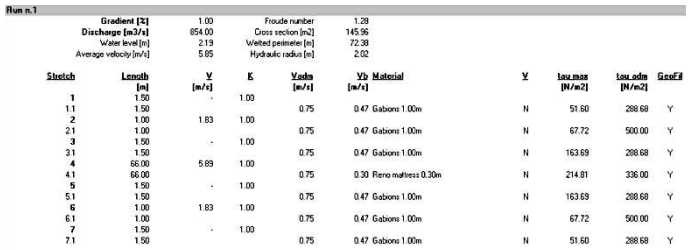
Lining	Thickness (m)	Stone Size Range (mm)	Recomm. d50 (mm)	Allowable Velocity (m/s)
Macmat R	blanket	-	-	1.5 - 3.0 (< 60hrs)
Geomac	0.23-0.30	Soil/rock	-	3.0 - 4.0 (< 60 hrs)*
Reno Mattress	0.17	75-100	85	4.2
	0.23	75-125	100	5.5
	0.3	100-150	125	6.4
Gabion	0.5	100-200	150	7.6

* values extrapolated from test results on Macmat R

Table 3. Allowable shear stress – typical values for tested erosion systems

Revetment Material	stone fill (mm)	Manning n. unveg	Unveg. $t_{allowable}$ (N/m^2)	Manning n. veg	Vegetated $t_{allowable}$ (N/m^2)
Macmat *unreinf (5–60 hrs)*	-	0.03	170 – 60	0.1	330 – 180
Macmat R *(5–60 hrs)*	-	0.03	190 – 80	0.1	350 – 200
Geomac (0.23-0.30m) *(5–60hrs)*	soil & rock	0.03	250 - 120	0.1	400 - 250
Reno mattress 0.17m	75 – 100	0.028	190 – 200	0.12	400
Reno mattress 0.23m	75 – 125	0.028	224 – 250	0.12	450
Reno mattress 0.30m	100 – 150	0.028	260 – 300	0.12	450
Gabions 0.50-1.00m	100 – 250	0.03	450	0.12	500

From the model analysed in MACRA 1, it can be seen in Figure 4 that the allowable velocity and admissible shear strength of the 300mm Reno Mattress are met. The channel thus required a minimum lining of a 300mm Reno Mattress to the point where the formalization of the channel concludes.



Run n.1

Gradient [%]	1.00	Froude number	1.28
Discharge [m3/s]	854.00	Cross section [m2]	145.96
Water level [m]	2.19	Wetted peimeter [m]	72.38
Average velocity [m/s]	5.85	Hydraulic radius [m]	2.02

Stretch	Length [m]	V [m/s]	K	Vadm [m/s]	Vb [m/s]	Material	V	tau max [N/m2]	tau adm [N/m2]	GeoFil
1	1.50	-	1.00							
1.1	1.50			0.75	0.47	Gabions 1.00m	N	51.60	288.68	Y
2	1.00	1.83	1.00							
2.1	1.00			0.75	0.47	Gabions 1.00m	N	67.72	500.00	Y
3	1.50	-	1.00							
3.1	1.50			0.75	0.47	Gabions 1.00m	N	163.69	288.68	Y
4	66.00	5.89	1.00							
4.1	66.00			0.75	0.30	Reno mattress 0.30m	N	214.81	336.00	Y
5	1.50	-	1.00							
5.1	1.50			0.75	0.47	Gabions 1.00m	N	163.69	288.68	Y
6	1.00	1.83	1.00							
6.1	1.00			0.75	0.47	Gabions 1.00m	N	67.72	500.00	Y
7	1.50	-	1.00							
7.1	1.50			0.75	0.47	Gabions 1.00m	N	51.60	288.68	Y

Figure 7. Results from MACRA 1.

MACRA 1 also provides the designer with a long section view of the channel as shown in Figure 5 below.

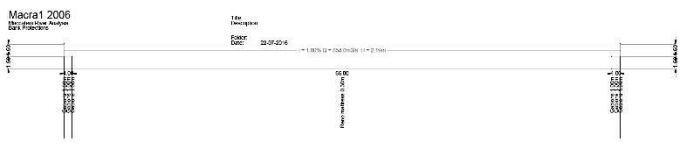


Figure 8. Section view from MACRA 1.

7 CONCLUSIONS

The use of gabions and reno mattresses as a substitute where there is no rock foundation present to construct a conventional concrete spillway overflow structure, is viable, provided that the design is implemented as per specifications.

Performing a proper geotechnical investigation as part of the detail design phase is vital to prevent surprises during construction.

Appointing a contractor with adequate experience on construction of gabions and reno mattresses is very important to save time and costs.

8 REFERENCES

Chanson, H. 2000. Forum article. Hydraulics of stepped spillways: current status, *J. Hydr. Eng., ASCE*. 126(9): 636-637.

Chanson, H. 1994. *Hydraulic design of stepped cascades, channels, weirs and spillways*. Pergamon: Australia.

Di Pietro, P. 2000. *Soil Bioengineering and ecological systems: Learning together how to handle erosion problems in natural and altered habitats*.

Federal Highway Administration 1975. Hydraulic design of energy dissipators for culverts and channels, Washington.

Fortier & Scobey. 1926. *Fundamentals of Engineering Design. Permissible Velocities*.

Moore, W.L. 1943. Energy loss at the base of a free over fall. *Trans. ASCE*. Vol. 108.

Rand, W. 1955. Flow geometry at straight drop spillways, Paper 791 ASCE, Vol. 81.

SANCOLD 1991. Guidelines on Safety in Relation to Floods. Report No 4

Wuthrich & Chanson. 2014. Aeration Energy Dissipation Over Stepped Gabion Spillways: A Physical Study.