THE USE OF MECHANICALLY STABILIZED EARTHFILL IN THE RAISING OF EARTH DAMS

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ABSTRACT

Mechanically stabilized earthfill (MSE) is well known and widely used in road construction, but is a relatively new concept in the South African dam industry. First used for the raising of Rietvlei Dam in 1989, the local use of MSE has had a slow start. More recently the Department of Water Affairs has embarked on rehabilitating a number of earth dams with inadequate spillway capacity, using MSE for raising the crest of three dams. The use of MSE compared to more conventional methods of raising is discussed, with specific reference to the three dams as well as Rietvlei Dam.

1. INTRODUCTION

The current form of mechanically stabilized earthfill (MSE) was developed in the early 1960's by Frenchman Henri Vidal and the patented concept quickly found wide application in the civil engineering industry. Since the development thereof it has gradually changed urban landscapes across the globe as it proved to be a cost effective alternative to conventional concrete retaining walls, whilst still being aesthetically appealing. In South Africa the patent license is held by Reinforced Earth (Pty) Ltd and is therefore locally very often simply referred to as reinforced earth. To avoid confusion with the license company the term 'mechanically stabilized earthfill' (MSE) will be used hereafter.

Mechanically stabilized earthfill is well known and widely used in local road construction, specifically road cuttings and bridge abutments, but it is a relatively new concept in the South African dam industry. First used for the raising of Rietvlei Dam in 1989, the local use of MSE in dam construction has had a slow start. More recently the Department of Water Affairs (DWA) embarked on rehabilitating a number of earth dams with inadequate spillway capacity, using MSE for raising the non-overspill crest (NOC) of three dams. These dams are Vaalkop Dam, Gcuwa Dam and Kromellenboog Dam.

In 2005 the DWA identified a significant number of dams with dam safety related problems, 80 % (Muller: 2007) of which had insufficient spillway capacity. For all dams which have been rehabilitated or where rehabilitation work is currently underway, detailed option analyses were undertaken to find the best solution to address the identified problems. In the case of insufficient spillway capacity, raising of the NOC or raising of the NOC in combination with alterations to the existing spillway proved to be the most cost effective solution. The use of MSE for raising the non-overspill crest of a dam compared to more conventional methods of raising is discussed hereafter, with specific reference to the three dams as mentioned above as well as Rietvlei Dam.

2. MECHANICALLY STABILIZED EARTHFILL: WHAT IS IT ?

Mechanically stabilized earthfill can be described as compacted granular earthfill material containing metal or synthetic reinforcement strips at regular intervals to improve the stability thereof. It is typically used to create a steep earth slope or vertical earth face. The reinforced earth mass forms a composite gravity retaining structure that can be engineered to meet specific loading requirements. The reinforcing strips are tied to facing elements on the outer face of the slope or wall. The stability of the structure is obtained and dependant on the strength of the reinforcing strips and the friction between the strips and the fill material. The facing elements may take the form of concrete panels or concrete blocks. They contain the reinforced fill material and provide protection against erosion.

Figure 1 shows a typical MSE arrangement as used for the raising of the non-overspill crests of the departmental dams previously mentioned.

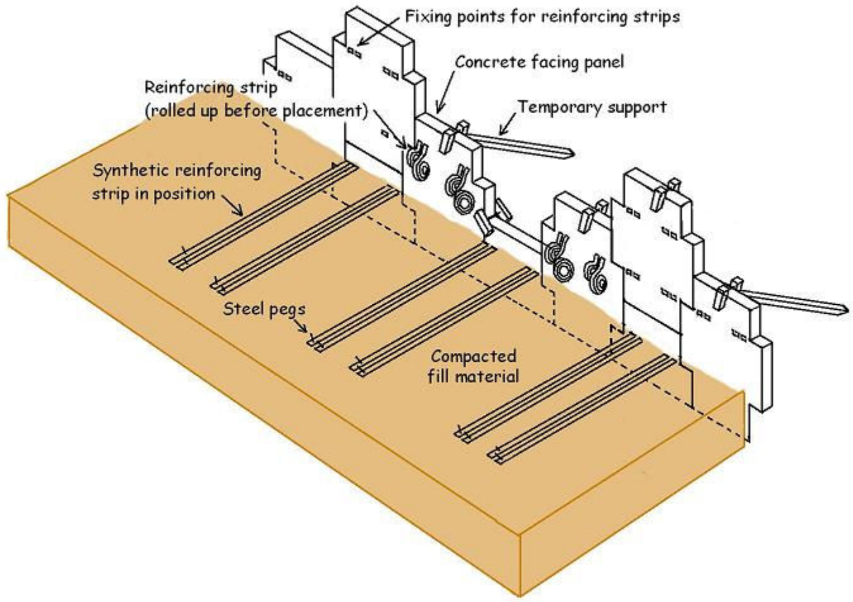

Figure 1 Typical MSE Arrangement

3. OPTIONS ANALYSES

3.1 General Approach

In order to assess the applicability and financial merits of MSE as a means of dam wall raising against other methods the designer needs to adopt and follow a defined and systematic design approach. The preliminary design stage is probably the most important stage as far as the cost of the final works is concerned. During this stage various options are considered and the costs of feasible options are compared. A typical options analysis in its basic form should have two distinct sequential stages, namely a ‘fundamental’ stage and an ‘analysis’ stage. The purpose of the first stage should be to obtain all relevant information which may impact on the design and to quantify all the ‘non-negotiables’ as far as possible. In the case of dam rehabilitation these aspects may vary significantly but may typically include requirements or preferences from the client or dam operator, physical constraints on site like limited working areas or restricted access, issues surrounding expropriation of land and social and environmental requirements. During this stage certain options will automatically be eliminated.

In the ‘analysis’ stage the designer lists the remaining feasible options and do a cost comparison. Where alterations to the spillway or raising of the NOC has in principle been identified as feasible rehabilitation options (‘fundamental’ stage) for dams with insufficient spillway capacity, the designer would typically follow an iterative process (‘analysis’ stage) to determine the most cost effective solution. In the case of earthfill embankment dams alterations to the spillway would normally involve an increase in the effective spillway length. The estimated cost of increasing the spillway length should be weighed against the cost of raising the NOC and combinations of these two options should be priced to find the most cost effective solution. This should be done for different methods of raising but within the framework of restrictions imposed during the ‘fundamental’ stage.

It should be borne in mind that when comparing different options of raising, it is not only the cost of the raising itself that should be considered but all costs resulting from that specific type of raising. In many instances these ‘secondary’ costs would be very similar and primarily a function of the height rather than the type of raising. It is preferable though to calculate these costs independently for each method to avoid any surprises.

Some of the more general ‘secondary’ design aspects which should be taken into consideration when doing a cost comparison for a raising are the following:

- The stability of the embankment slopes under the new imposed (dead) load of the raising.
- Increased hydrostatic loading on structures during design and extreme flood conditions.
- The necessity for raising spillway training/retaining walls.
- The increased unit discharge in the spillway and the erosion resistance of the spillway channel.
- Provision for future embankment settlement under the newly imposed load.
- Detail of raising at the abutments and beyond the existing embankment above the existing NOC.
- Access requirements.
- Cost of maintenance.
- Post-construction public safety / safety of operating officials.
- Post-construction monitoring and inspection requirements.

The graph in Figure 2 illustrates the results of a hypothetical cost analysis and optimization for an embankment dam with a bywash spillway. This analysis is for a MSE raising, but similar analyses will have to be done for other methods of raising. Some relevant results of the (hypothetical) ‘fundamental’ analysis are incorporated into this analysis.

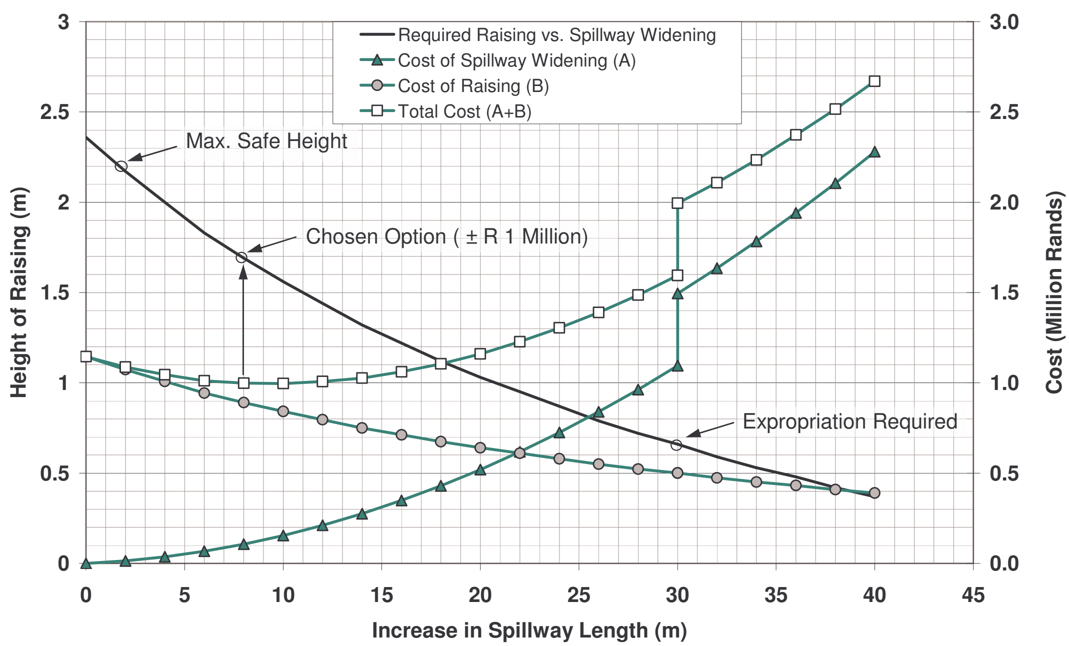

Figure 2 Cost Analysis and Optimization for Raising with MSE

From Figure 2 it is clear that for the optimum cost of R 1 million the bywash spillway must be widened by 8 m and the crest of the embankment dam raised by 1.7 m.

3.2 Mechanically Stabilized Earthfill: Fundamentals and Cost Estimation

There are generally very few fundamental constraints when raising embankment dams with MSE. One very important constraint however is that in all cases there would be a limit on the height with which the embankment can be raised due to slope stability reasons. A slope stability analysis should be conducted from which the allowable maximum height of raising is determined (see Section 4). In cases where the safe raising height is less than the required height, MSE may be used in combination with a parapet wall.

The remainder of the fundamental constraints needs to be quantified on a site specific basis. The selection and testing of a suitable earthfill material may not be considered part of the 'fundamental' process, but semantics aside, a late change in the source of fill material can potentially derail an otherwise financially sound project. A borrow area for semi-pervious material or sand must be available as well as the required permits for the use thereof. If not, the use of crushed stone (sand) may be necessary. The suitability of the proposed fill material should be confirmed by laboratory testing before finalizing a cost comparison of the various raising options.

In principal MSE can be constructed over either earthfill or rockfill, but rockfill will require transition layers with specific grading criteria. If required MSE can be designed as a submerged structure, in which case specific design criteria will apply. The facing panels are both structurally functional and aesthetic and can be customized if required. The crest width can be altered, but a minimum width is required to accommodate compaction of the granular earthfill material. The new crest can be made wider than the existing crest if required. A MSE wall may be constructed on one side of the crest (upstream or downstream) whilst raising the opposite side with conventional (sloped) earthfill material. The need for possible future raisings of the Full Supply Level (FSL) and NOC to increase the yield of the reservoir may in some instances form part of the 'fundamental' stage and can dictate the method for the current raising of the NOC.

The main three components contributing to the cost of raising a dam wall with MSE is the pre-cast concrete facing panels, reinforcing strips and the granular earthfill. Allowance should also be made for erection of the panels and a concrete footing for the panel walls. The cost of the facing panels and reinforcing strips can be readily obtained from the supplier, whereas the cost of the granular fill material will primarily depend on the distance between the source and the site and whether a commercial source is used or not. For the four dams mentioned earlier a subcontractor was used for the design of the MSE part of the works as well as the supply of reinforcing strips and concrete panel moulds. When using previous pricing data for the purpose of cost estimation it should be borne in mind that unit rates for MSE normally already include the subcontractor's markup. However, site establishment cost for the MSE subcontractor should be taken into account.

3.3 Case Studies

3.3.1 Rietvlei Dam

For the rehabilitation of Rietvlei Dam near Pretoria a physical hydraulic model study of the bywash spillway was conducted to help in the comparison of various options. The options included lowering the FSL with 2.2 m, raising the crest, widening the spillway and combinations of the latter two. Lowering the FSL would have reduced the effective storage by 31% and was not considered a long term feasible solution. It was concluded that widening the spillway and raising the NOC with approximately 3 m would be the best option. The design for the raised wall required that there be an amount of flexibility in the completed structure and the designers found the MSE method of retaining wall construction ideally suited for this purpose.

3.3.2 Vaalkop Dam

Vaalkop Dam near Brits was raised in 2007 using MSE on the crests of the dam's earth embankments. The service spillway of the dam comprises of a ± 27 m high concrete gravity structure and is flanked by a 520 m long and a 1 260 m long earth embankment on the left and right flank respectively. The original dam had a 430 m long fuse plug type auxiliary spillway on the right abutment.

It was realized from the onset that modifications to the existing service spillway to increase the spillway capacity would not be cost effective. Based on the options analysis conducted during the preliminary design stage it was concluded that raising the crest of the embankment(s) and closure of the auxiliary spillway (by raising the embankment thereof) would be the best option. This option solved the problem of property and lives at risk in the auxiliary spillway's flood plain and the risk of overtopping of the main embankment should the auxiliary spillway embankment not breach as required. Based on flood routing analysis it was concluded that the main embankment(s) needed to be raised with a minimum of 1.08 m and the auxiliary spillway embankment with at least 1.42 m. Two options for the raising were identified, namely:

- Raise the crest of the embankments with soil and provide MSE with concrete panels on the upstream side (decreasing the crest width from 7.5 m to 6.4 m) and raise the auxiliary spillway embankment with conventional earthfill.

- Raise the crest of the embankments and the auxiliary spillway embankment with a concrete parapet wall.

Cost estimates of the two options carried out during the preliminary design stage showed that they are within 5 % of each other, with the MSE option being the cheaper of the two. Apart from the cost, the MSE option was recommended on the basis that it would accommodate settlement (and therefore stability best) and will suit the visual environment best.

3.3.3 Gcuwa Dam

Prior to the raising in 2009, Gcuwa Dam near Butterworth comprised of a 251 long earth embankment with a maximum height of 18.5 m and an ogee shaped concrete gravity spillway on the right flank. The options considered to increase the flood handling capacity involved raising the NOC, the provision of a new auxiliary fuse plug spillway and raising the NOC in combination with the provision of a new spillway or alterations to the existing spillway. For the raising of the embankment the following options were considered:

- MSE with pre-cast concrete panels on the upstream and downstream side.

- Concrete retaining walls upstream and downstream with earthfill in-between.

- Parapet wall on the upstream side with a wide footing.

The estimated cost of providing a new 160 m long auxiliary fuse plug spillway on the left flank compared to the cost of raising the embankment with the required 2.5 m by means of MSE and widening the existing spillway by 6 m were found to differ with less than 1%, with the auxiliary spillway being the cheaper option. This option however contained relatively high risks as far as the time of implementation is concerned as it required detailed geotechnical investigations of the proposed fuse plug foundation material and possibly compensation to affected land owners in the flood plain. With the required geotechnical investigation it was estimated that the cost of this option would in any case become more than the MSE option.

Raising the embankment crest by means of MSE with 2.5 m and widening the spillway with 6 m was recommended as the option to implement. The estimated cost of this option included demolishing of the existing spillway bridge and providing a new raised bridge, a concrete stability enhancement section on the downstream face of the existing gravity spillway and a raised outlet structure. For lengthening of the embankment beyond its existing NOC point on the left bank, a conventional embankment was found to be cheapest.

3.3.4 Kromellenboog Dam

The embankment crest of Kromellenboog Dam near Groot Maricopoort was raised in 2009 using MSE. Before the raising the dam wall comprised of a 485 m long earth embankment with a maximum height of 20.5 m. It has a 126.5 m wide bywash type spillway on the right flank.

The options considered to improve the flood handling capacity comprised of raising of the NOC level, widening of the bywash spillway on the right bank and providing an additional spillway (emergency spillway) on the left bank. It was however realized during the initial design stages that an emergency spillway would for various reasons be very expensive and this option was therefore discarded. The cost of raising the earthfill embankment, widening of the bywash spillway and combinations of these two options to achieve the required flood handling capacity were subsequently determined. Widening of the spillway would have required excavation in hard rock and blasting. Retaining the existing spillway width and raising the NOC by 2.1 m proved to be the most cost effective solution.

Two options for raising of the non-overspill crest were identified, namely adding rockfill on the downstream side and on the crest of the embankment and raising the crest of the embankment only by means of MSE. Cost estimates showed that the option of raising by MSE was cheaper by over R 2 million.

4. DESIGN AND SPECIFICATIONS

4.1 Internal and External Stability

The design of reinforced earthfill is divided into two main parts, namely design for external stability and design for internal stability.

External stability considers the behaviour of the site under the loading of the reinforced earthfill structure and is primarily influenced by in-situ conditions. The safety of the structure with regards to sliding, overturning and bearing pressure on the foundation is generally evaluated. The minimum reinforcement length as determined in the external stability analysis will normally prove to be the minimum reinforcement length of the structure.

In the case of a raised earth embankment the reinforced earthfill is an imposed load on the existing structure and would invariable lead to a reduced factor of safety with regards to slope failure. It is of paramount importance that a detailed slope stability analysis of the existing embankment with the reinforced earthfill be conducted as part of the preliminary design. For this purpose a geotechnical investigation should be conducted to determine the composition of the existing embankment and the shear strength parameters of the existing embankment materials. The designer should ensure that the factors of safety as obtained in the analysis still comply with the set criteria. The geotechnical analysis should be done at the onset of the preliminary design stage in order to ascertain whether reinforced earth can be used and if so, what the height restriction is. The safe raising height would depend on the results of the slope stability analysis.

Internal stability calculations deal with the interrelationship between the components of the reinforced earthfill structure itself; the facing elements, reinforcing strips and the earthfill material. The reinforcement length (as obtained in the external stability analysis) must be sufficient to provide the minimum required factor of safety against pullout. The required number (density) of strips is calculated so that the cumulative cross sectional area of the strips is sufficient to carry the required (factored) tensile stress and strip rupture is avoided. The design of the strips therefore depends on the tensile strength of the strips and their adherence capacity. The South African National Standards document "SANS 207:2006, Edition 1: The Design and Construction of Reinforced Soils and Fills" provides formulae and principles for internal stability calculations.

4.2 Specifications

In part 7200 of the "Standard Specifications for Road and Bridge Works for State Road Authorities", hereafter referred to as COLTO (COLTO: 1998), detailed standard specifications for MSE are provided in accordance with the patented method registered under the trade name of REINFORCED EARTH. The specifications cover the type and quality of steel reinforcing strips, cladding panels, fixing elements associated with the panels as well as construction and quality control specifications.

Backfill material for MSE may be a natural soil or crushed rock. Irrespective of the origin thereof, all selected backfill material must be well drained, be durable, not break down or change its properties during construction, not be prone to post-construction creep and not contain any organic material (Kalili: 2008). In COLTO suitable material is specified in terms of particle size distribution, Plasticity Index and chemical and electro-chemical properties. Granular fill ranging from fine sand to pebbles smaller than 200 mm is generally considered suitable. Silts and clays with a Plasticity Index less than 30 is referred to as "intermediate conditionally suitable" material and may only be used if approved by the engineer. The chemical and electro-chemical specifications in COLTO are to limit corrosion of galvanized steel reinforcing strips.

As maximum dry densities (MDD) for earthfill in dam embankments is typically determined by means of the Standard Proctor method, the designer should take note that COLTO gives the compaction specification for MSE in terms of a modified AASHTO density. For many projects compaction of earthfill other than MSE may also be required, for instance where normal fill is used adjacent to MSE or lengthening of the embankment by means of conventional earthfill. Where both the Standard Proctor and modified AASHTO densities are used in a project, the specifications and drawings should be clear and consistent with regards to which compaction criteria applies to which part of the works.

An aspect not covered by COLTO (COLTO: 1998) is the use of synthetic reinforcing strips. These strips were used for the raising of Vaalkop Dam, Gcuwa Dam and Kromellenboog Dam. The Freyssisol type synthetic strip is currently widely used by Reinforced Earth (Pty) Ltd. These strips comprise of polyester fibers encased in a polyethylene sheath. The cost of these strips are similar to that of steel strips, but the advantage of using synthetic strips is that they are not sensitive to corrosive conditions, especially when combined with fully synthetic connections to the panels such as the "Omega" type. Synthetic strips would therefore be the preferred option when raising embankment dams with MSE.

Case Studies: Specific Design Detail

Figures 3 to 6 hereafter shows typical details of the MSE works for the raising of Rietvlei Dam, Vaalkop Dam, Gcuwa Dam and Kromellenboog Dam respectively. It can be seen that for all four dams suitable safety barriers were provided on the crest. MSE on embankment crests invariably forms a vertical or near vertical drop and it is therefore essential that some sort of barrier be provided for safety reasons, even if the crest is only accessed by operating personnel. Another typical design detail included in each case (but not shown in the figures) was the placement of geotextile strips at all horizontal and vertical concrete panel joints, between the panels and the earthfill material to prevent migration of soil. The geotextile was glued to the concrete panels. Where two or more panels were placed on top of each other, a tube and dowel system was used to help keep the panels in their relative positions. To allow (vertical) flexibility and avoid abrasion, rubber pads with serrated edges were used as vertical spacers between the panels.

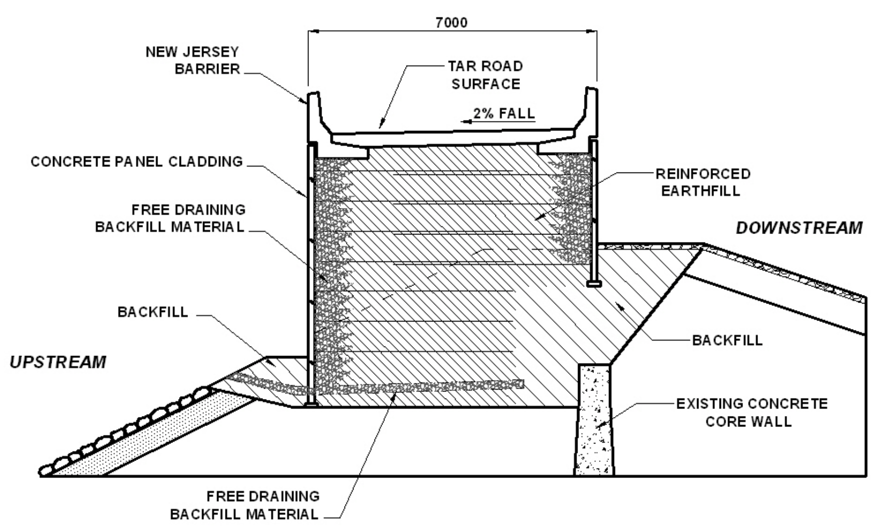

Figure 3 Rietvlei Dam: Typical Detail of MSE Crest

At Rietvlei Dam the versatility of the MSE concept for raising embankment dam crests are well illustrated. Noteworthy is the offset between the old and the new centre-line of the crest, with the new crest moved upstream. Specific detail was incorporated to ensure that the MSE is totally free draining. A 500 mm thick layer of specified free draining material was placed immediately behind the concrete cladding panels. New Jersey type barriers were chosen as a public road runs along the crest.

At Vaalkop Dam the existing crest was relatively wide and it was therefore possible to raise the crest to the required level by using MSE only on the upstream side whilst maintaining the existing slope on the downstream side. The minimum requirement for the engineering properties for the earthfill was PI < 15, Φ > 30° and C ≈ 0 kPa. For ease of construction no distinction was made between the reinforced fill and conventional fill adjacent to it in terms of layering or compaction. It was specified that both be compacted to 98% of the Standard Proctor Maximum Dry Density. The existing internal drainage system was utilized to drain the MSE.

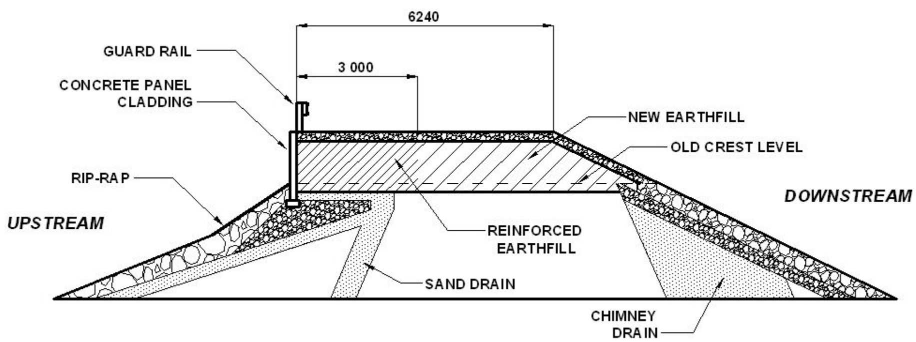

Figure 4 Vaalkop Dam: Typical Detail of MSE Crest

For external stability purposes the MSE structure at Gcuwa Dam was founded approximately 1 m below the existing crest level. During construction it was found that the top part of the existing embankment material was compacted below standards, with values ranging from 86% to 91% of the Standard Proctor maximum dry density. Tri-axial testing, however, indicated that the shear strength values are still within the (conservative) design assumption range.

As in the case of Rietvlei Dam, specific attention was given to internal drainage at Gcuwa Dam. As mentioned earlier, the stability of the MSE structure relies on friction between the reinforcing strips and the earthfill. A build up of pore pressure within the MSE should be avoided as this may reduce the friction considerably. In the case of Gcuwa Dam the (free draining) reinforced earthfill is underlain by impervious clay material and it was essential to provide adequate drainage measures. This was done in the form of uPVC drain pipes at 30 m intervals. Below the MSE the uPVC pipes were slotted to allow ingress of water into the pipes.

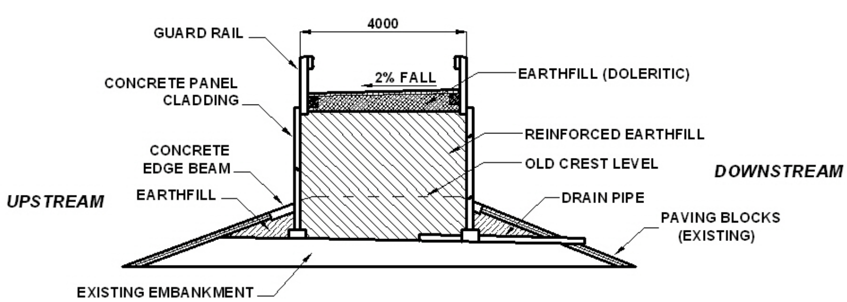

Figure 5 Gcuwa Dam: Typical Detail of MSE Crest

Consideration was initially given to source the fill material for the raising of Gcuwa Dam from a borrow area to the right of the spillway's approach channel or from a picnic area on the left flank of the dam. Although PI values of around 8 % was obtained in some of the test pits, these materials generally contained too little sand as required for providing the shear strength between the synthetic reinforcing strips and the soil. For the MSE a suitable crusher dust was therefore sourced from a commercial quarry.

Apart from the tip of the impervious clay core, the MSE at Kromellenboog Dam was underlain by pervious to semi-pervious zones and no additional drainage measures were therefore considered necessary. The total crest width of 7.62 m was retained, giving an effective width of 6 m between the guardrails for vehicle movement. Like the previous 3 cases, a crossfall of 2% was provided in the upstream direction for surface drainage.

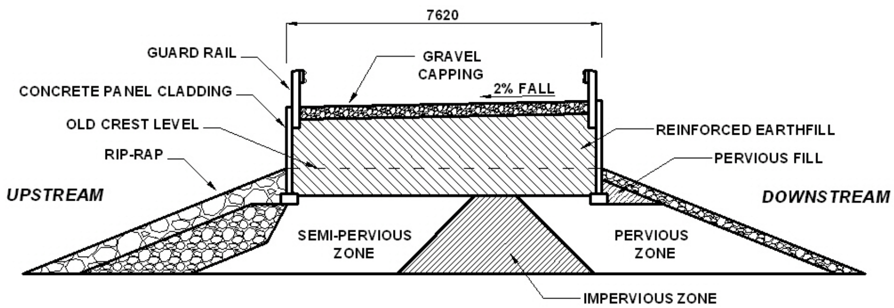

Figure 6 Kromellenboog Dam: Typical Detail of MSE Crest

5. CONSTRUCTION

5.1 General

For all dams discussed here Reinforced Earth (Pty) Ltd was appointed as subcontractor for the MSE part of the works. This involved the design for internal stability of the MSE, supply of the necessary moulds for the pre-cast panels and the supply of the reinforcing strips and fixing elements. The sub-contractor also provided technical assistance with regards to the erection of the pre-cast panels, placement of the strips and compaction of the earthfill.

5.2 Case Studies: Problems and Issues

The most notable problem during construction at three of the dams proved to be the quality control with regards to the manufacturing of the concrete panels. At Rietvlei Dam a total of 40 precast concrete panels were condemned during the contract period (Stewart Scott: 1992). The problem with regards to the quality of the precast panels surfaced again at Gcuwa Dam. It was found that (again) 40 of the stacked precast panels had cracked quite severely. Some repair work was undertaken but was not effective. The long term integrity of these panels was in question and they were subsequently rejected. It was suspected that the primary cause of the cracking was a combination of a high turnover through the precast moulds and the method of stacking the panels once removed from the moulds. The problem with cracking of the precast panels also occurred at Kromellenboog Dam, but to a much lesser extent. Minor cracking were observed in a few panels but after investigation thereof it was decided that these could be used at certain non-critical locations.

At Rietvlei Dam a large amount of time was spent in obtaining suitable graded filter material for the MSE part of the works. This had a significant impact on the works. A minor issue was a change of the finish of the concrete cladding panels from an exposed aggregate surface to an off-shutter surface as there was doubt whether the panels could be manufactured with a uniform finish.

At Gcuwa Dam some problems were experienced with over compaction of the crusher dust which was used for the MSE. On request of the contractor the moisture content specification for compaction of the crusher dust was changed to OMC – 4% which gave acceptable results. The Contractor further experienced some difficulty to get the top edges of the upper panels perfectly aligned. The original design made provision for a concrete coping which generally hide any misalignment, but for cost reasons this was discarded. After deliberation it was felt that the presence of the guardrails would soften the visual effect of any skewness of the top edges and the overall appearance was considered acceptable.

5.3 Practical Considerations

Figure 7 and Figure 8 below show construction of the MSE works at Gcuwa Dam. Because of the confined width, hand operated compaction equipment were used. Where the working area is wide enough a smooth drum roller is generally suitable. The contractor however should avoid using heavy compaction equipment too close to the facing panels, as this has the tendency to push the reinforcing strips towards the facing panels. As successive layers of earthfill are compacted, the facing panels are pushed slightly outwards by the fill. To compensate for this the panels are not positioned completely vertical at the start but is actually slightly inclined inwards, being suspended with tie rods at the top.

In many instances where synthetic strips are used, a shallow trench, typically about 750 mm wide by 150 mm deep, is dug parallel to the facing panels some distance away. The reinforcing strips are placed across it, tensioned (by hand) and pegged down. When the next layer of fill is compacted over it, the reinforcing strip is pushed downwards into the trench, tensioning it and pulling the facing panels towards the fill. This method allows the use of heavier compaction equipment near the facing panels.

When installing guard rail posts the contractor should be careful not to damage the top reinforcing strips. For this purpose it is better to first install the posts and then drill the required holes in the guard rail at specific locations. The posts may not always be evenly spaced.

Figure 7 Gcuwa Dam: Compaction of MSE

Figure 8 Gcuwa Dam: Excavated trench

6. POST-CONSTRUCTION PERFORMANCE

Of the four dams discussed here it is only the Rietvlei Dam which can really be used to evaluate the long term performance of the MSE method of raising as all the other dams have only recently been completed.

During the second dam safety inspection by Stewart Scott Consulting Engineers in 1996, the condition of the reinforced earth walls were found to be excellent with no indication of movement, deflection or settlement. On recommendation of the inspection report, Reinforced Earth (Pty) Ltd carried out pull-out tests of special steel test strip sections to determine the durability of the Reinforced Earth 40 x 4 mm mild steel strips used to anchor the concrete panel walls. The conclusion of the tests was that the strips are performing very satisfactory (Stewart Scott: 1999).

During the most recent dam safety inspection (Badenhorst: 2005) no cracks were observed on the bitumen surfaced crest of the dam. The lower part of the concrete walls was inspected very carefully for movement or cracks but no sign could be identified. At the time of the inspection there was no instrumentation to monitor settlement of the crest. From visual observations during the inspection it was concluded that there where no apparent problems with or relating to the Reinforced Earth part of the embankment.

7. CONCLUDING REMARKS

In the recent design of rehabilitation works for three dams of the Department of Water Affairs, raising the earth embankments of these dams using MSE were compared to various other rehabilitation alternatives and chosen as the preferred solution. Increasing the spillway capacity of dams cost effectively requires detailed options analyses and each alternative needs to be evaluated on its merits. The raising of Rietvlei Dam back in 1989 and the recent dam raisings by the Department of Water Affairs have established MSE as a viable option (for raising earth embankment dams) in the South African dam industry.

8. ACKNOWLEDGEMENTS

With the exception of Rietvlei Dam, the bulk of the material on the case studies above was sourced from design reports for the various raisings, prepared by BKS (Pty) Ltd for the Department of Water Affairs. The author wishes to thank Mr. D Badenhorst and Mr. R Steenkamp of BKS (Pty) Ltd for providing supplementary information. Reinforced Earth (Pty) Ltd is thanked for the supply of technical data.

9. REFERENCES

Badenhorst, D.B. (2005). Rietvlei Dam, Safety Inspection Report. Prepared by BKS (Pty) Ltd for the City of Tswane Municipality, July 2005.

COLTO (1998). Standard Specifications for Road and Bridge Works for Road Authorities, prepared by Committee of Land Transport Officials (COLTO), published by South African Institution of Civil Engineering (SAICE), 1998.

Mulder, H. (2007). Rehabilitation of State Dams in South Africa. Proceedings of a Three Day Short Course on Dam Design with the focus on Hydropower and Environmental Aspects, pp 304-317.CSIR Conference Centre, Pretoria, June 2007.

Kalili, N (2008). Reinforced Earth Design and Application. Course notes for Applied Geomechanics (GO3), Department of Civil Engineering, University of Stellenbosch. April 2008.

Stewart Scott (1992). The Rehabilitation of Rietvlei Dam, Construction Report. Prepared by Stewart Scott Consulting Engineers for Pretoria City Council, June 1992.

Stewart Scott (1999). Rietvlei Dam: Rehabilitation of Dam, Summary of Dam Safety Aspects. Prepared by Stewart Scott Consulting Engineers for Pretoria City Council, June 1999.