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Reinforced Rockfill for Construction Flood Control

By Philip J. N. Pells[1]

Introduction

Three dams (Bridle Drift, Lesapi, and Xonxa) have been constructed in southern Africa making use of downstream zones of reinforced rockfill to control major construction floods. In the cases of the Bridle Drift and Lesapi dams relatively minor failures occurred when floods passed over the partly constructed embankments, while at Xonxa a major failure occurred.

In a paper presented at the 10th Large Dams Congress at Montreal (2), the experiences at Bridle Drift were described in detail and the design of the then proposed Xonxa dam was considered in the light of these experiences. The present paper first briefly summarizes the Bridle Drift experiences and then records what happened during the construction of the Xonxa dam. Mention is made of the problems experienced at the Lesapi dam in Rhodesia which, unfortunately, only became known after the failure at Xonxa. From these failures and from the successful performance of the Googong dam, recently completed near Canberra, conclusions are drawn regarding the important factors to be considered in the design and construction of reinforced rockfill dams of this type.

Bridle Drift Dam

As shown in Fig. 1, the Bridge Drift dam is a sloping core rockfill structure constructed originally to a height of 167 ft (51 m), and subsequently raised

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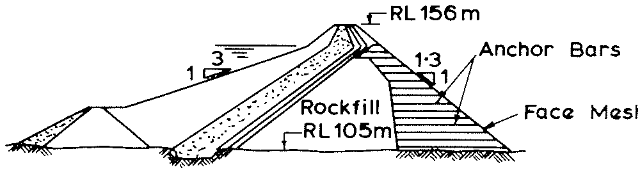
to 177 ft (54 m). The downstream rockfill zone, constructed ahead of the core and upstream zone, is composed of dolerite from the 180-ft (55-m) deep spillway excavation. At the time when the major flood occurred the downstream rockfill had reached a height of 50 ft (15 m) above river bed level but construction of the core had not yet commenced.

The downstream rockfill zone was placed in 3-ft to 5-ft (1.0-m to 1.5-m) layers, watered and compacted by 10-ton (9,100-kg) vibrating rollers. The rockfill reinforcement was based on the practice current in Australia at the time (3,4), but modified on the basis of model tests. The reinforcement thus consisted of two layers of quite light weldmesh on the downstream face, anchored back with horizontal bars at 10-ft (3-m) vertical intervals. Very heavy anchorage was provided because at the time the big concern was with the possibility of deep seated sliding during overtopping. Thus the lower anchor layers consisted of 1-1/2-in. (38-mm) diameter bars at 9-in. (230-mm) centers. The spacing these anchor bars increased in the higher layers while above a height of abo... 100 ft (30 m) the anchor layers consisted of 3/4 in. (19-mm) bars at 5-ft (1.5-m) centers. The surface weldmesh was made up of 0.28-in. (7-mm) diam wires horizontally at 6-in. (150-mm) centers and 0.2-in. (5-mm) diam wires down the face at 9-in. (225-mm) centers.



1m = 3·28ft

FIG. 1.—Bridal Drift Dam-Cross Section

The construction procedure adopted initially was to place the horizontal reinforcement, attach the mesh for the next 10-ft (3-m) lift and allow this to hang down over the completed fill. The 10-ft (3-m) lift was then completed and the mesh was drawn up tight over the face and attached to the next layer of anchor bars. The obvious fault in this system was that there remained substantial amount of unprotected rockfill on the embankment at any given time.

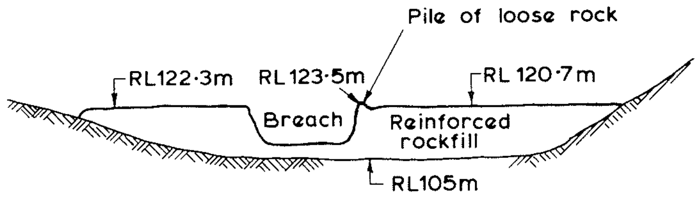
The major flood occurred overnight on April 10, 1967. The rockfill was fully reinforced to a height of 50 ft (15 m) above river bed level. However, the next 10-ft (3-m) lift was only partly completed, with the first 5-ft (1.5-m) layer having been compacted over the full width of the embankment and the ensuing 5-ft (1.5-m) lift placed but not compacted over the right half of the valley. At the end of this second lift, near the center of the valley, was a large pile of loose rock with its top about 5 ft (1.5 m) above the second lift.

The river started to rise rapidly at 6 p.m. and by the early hours of the next morning water was flowing nearly 13 ft (4 m) deep over the top of the completed reinforced rockfill, representing a flow of about 40,000 cu ft/sec (1,100 m³/s). The uncompacted rockfill on the right of the central pile was eroded and washed down the face, cutting the mesh and stripping the 0.28-in.

(7-mm) diam horizontal wires. This allowed erosion and sloughing of the rockfill in the face, a process that gradually worked back in the form of a channel. Further flow concentration thus occurred and by the time the flood subsided a breach had been carved about 33 ft (10 m) deep and 100 ft (30 m) wide, as shown in Fig. 2. About 30,000 cu yd (23,000 m³) of reinforced rockfill and 17,000 cu yd (13,000 m³) of unprotected rockfill were lost out of the total of 300,000 cu yd (230,000 m³) placed at that stage.

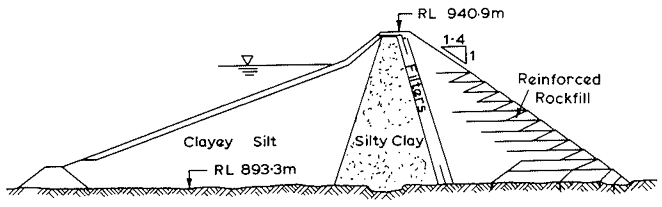
One other point of importance is that the lower, left hand section of the embankment, which in fact took the brunt of the flood, did not suffer much damage. Only near the center of the valley to the left of the large pile of rock had some of the compacted rock been eroded and washed down the face, cutting portions of the mesh in the lower layers.

The result of the failure at Bridle Drift was that considerable care was taken ...ring the subsequent construction to remove the possibility of loose rockfill



1m = 3·28ft

FIG. 2.—Bridal Drift-Downstream Elevation after Flood



1m = 3·28ft

FIG. 3.—Cross Section of Xonxa Embankment

being washed down the protected face and therefore damaging the weldmesh. The construction procedures are described elsewhere (2) and need not be repeated here. The Bridle Drift dam was completed without the occurrence of further major construction floods and thus the system was not tested. This carefully controlled construction sequence was written into the tender documents for the Xonxa dam for which also a reappraisal of the design of reinforced rockfill was undertaken.

Xonxa Dam

The Xonxa dam is a 158-ft (48-m) high composite earth and rockfill structure with a typical cross section as shown in Fig. 3. The dam is on the White Kei River in the Eastern Cape region of South Africa. The river has a small flow for most of the year [< 500 cu ft/sec (14 m³/s)] but is subject to sudden and violent floods at any time. Provision of bypass facilities for other than minor floods rendered an embankment dam uneconomical, but adopting the reinforced rockfill technique allowed the design of a very reasonably priced dam.

The site is at a point where the river flows through a very short gorge associated with a major dolerite intrusion. The river floor and lower portions of the right flank consist of massive fresh dolerite at the surface. However, the left flank can best be described as consisting of a classic dolerite residual profile as shown in Fig. 4. The problem was the difficulty in defining, from the limited borehole data available, at what depth one was out of residual "boulder" formation and into sound dolerite. This problem was to play a large part in the subsequent failure.

Design of Rockfill Reinforcement.—The design of the rockfill reinforcement is described in detail by Shand and Pells (2) and is only summarized here. In essence such a design must provide protection of the rockfill against: (1)

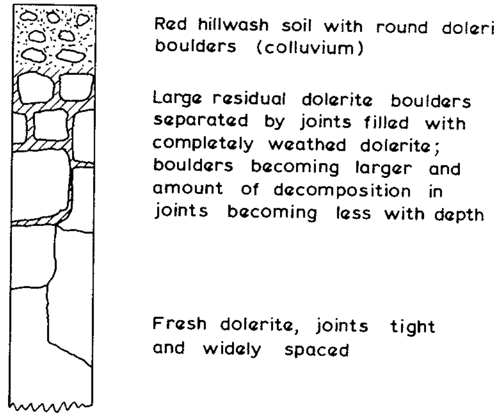


FIG. 4.—Idealized Geological Profile of Left Flank at Xonxa

Surface erosion and sloughing; and (2) deep seated sliding. Steel mesh held tightly over the surface provides protection against surface movement while anchor bars extending into the rockfill provide resistance against deep seated sliding.

Theoretical calculations show that only a light mesh is needed to prevent movement of surface rock associated with flow out of and down the rockfill face. The loading on the mesh is taken chiefly as "ring tension" in the vertical wires as the horizontal wires have little end restraint and serve to provide resistance against local bulging and to keep the vertical wires in place. The experience at Bridle Drift and also that reported from the Cethana Dam in Tasmania (1) indicated, however, the need for more than the theoretical amount of steel to be incorporated in the face mesh at Xonxa. This extra steel was required to provide protection against damage from rocks being dropped on the face during construction or washed down during overtopping. The mesh thus designed consisted of 0.4-in. (10-mm) diam wires at 15-in. (380-mm) centers

across the slope and 0.2-in. (5-mm) diam wires at 6-in. (150-mm) centers down the slope. The horizontal wires were required to be placed inside the relatively closely spaced vertical wires so as not to be easily knocked loose by debris falling down the slope.

The danger of deep seated sliding arises from the development of seepage pressures within the rockfill during the passage of a construction flood. It was realized during the design of Xonxa that with the strong layering that occurs in compacted quarry run rockfill it was possible that no significant seepage pressures would develop within the relatively short duration of a construction flood. If this were so then deep seated sliding would not be a possibility and the only reinforcement required would be a strong mesh on the downstream face tied back a short distance into the embankment. This aspect is covered further at the end of this paper but at Xonxa, as no quantitative information was available to determine the time required for seepage pressures to develop, long anchorage bars were provided assuming the development of full internal seepage pressures.

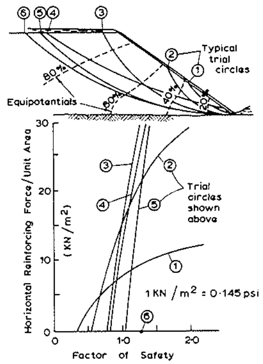


FIG. 5.—Determination of Anchorage Force for Stability Against Deep Seated Sliding

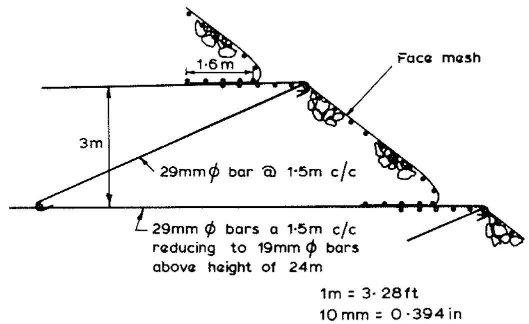


FIG. 6.—Reinforcing System Designed for Rockfill at Xonxa

The deep anchor bars at Xonxa were designed using a modified version of Bishop's simplified slip circle analysis (2). The procedure was as follows:

1. For a given embankment height the approximate flow net for the most critical flow condition (full overflow with zero tailwater depth) was determined taking into account the layering of the rockfill.
2. The unreinforced slope was analyzed using Bishop's simplified slip circle method to determine the shallowest circle, which without reinforcement had the desired factor of safety of 1.2. This determined the limit of the anchor bars, as deeper circles had a higher factor of safety.
3. The modified Bishop equation was then used to calculate the anchor force required to raise the factor of safety of shallow slip circles to the value of 1.2.

The preceding method is shown in Fig. 5. It was recognized at the time as providing only an approximate solution but appeared to be the best available approach.

The anchorage system designed for Xonxa is shown in Fig. 6. This involved a considerable decrease in the quantity of anchorage steel from that used at Bridle Drift. The heaviest layers of anchors consisted of 1-1/8-in. (29-mm) diam bars at 5-ft (1.5-m) centers, compared with 1-1/2-in. (38-m) diam bars at 9-in. (0.23-m) centers used at Bridle Drift.

When construction at Xonxa got underway the contractor considered that the oblique bars shown in Fig. 6 could not be placed satisfactorily and these were simply incorporated with the horizontal anchor bars.

Construction of Reinforced Rockfill.—Placement of the reinforced rockfill started early in 1971 after some delays associated with minor washaways of the small upstream cofferdam.

The surface mesh was made up in prefabricated panels. This was done so that the complete mesh protection for a 10-ft (3-m) lift could be erected along

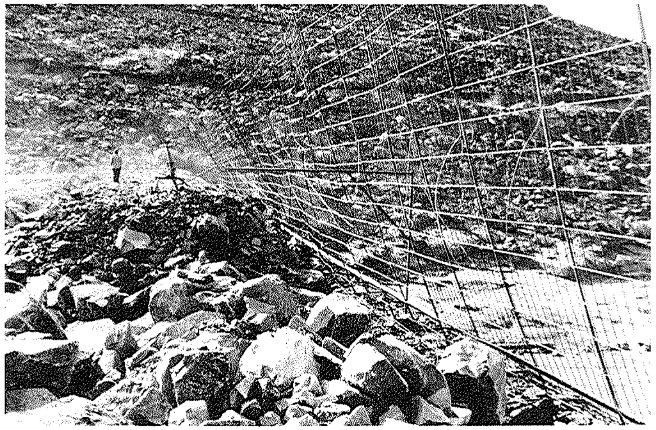


FIG. 7.—Rockfill Being Placed Behind Prefabricated Mesh Panels

the downstream edge of the previous layer prior to any further rockfill being placed, as shown in Fig. 7. The rockfill was advanced against the panels usually working from one flank, although it had originally been planned that work should proceed from both flanks. The idea of the prefabricated panels was that in the event of a flood occurring at any stage during the placement of a 10-ft (3-m) lift, the panels could be bent back and secured to the anchor bars lying on the preceding compacted layer. The layers were not in fact horizontal but sloped upward at about one in 20 in the downstream direction, thus minimizing the tendency for any overtopping flood to transport rock over the protected downstream slope. Although a fair amount of hand work was involved in the final finishing of the downstream face, the construction system worked well.

Fig. 8 shows the reinforced rockfill prior to the major flood. There was, however, one problem associated with the reinforcement, which was what to do at the flanks. As with most embankment dam design the reinforcement at Xonxa was designed in terms of a two-dimensional slice taken through the embankment in the center of the valley. Sufficient consideration was not given

during the design to the three-dimensional problem where the downstream face meets either valley side. On the site the problem was "solved" by suitably bending the panels under the ends of each layer. On the right flank the panels at the end of each layer were placed on and doweled to the fresh dolerite

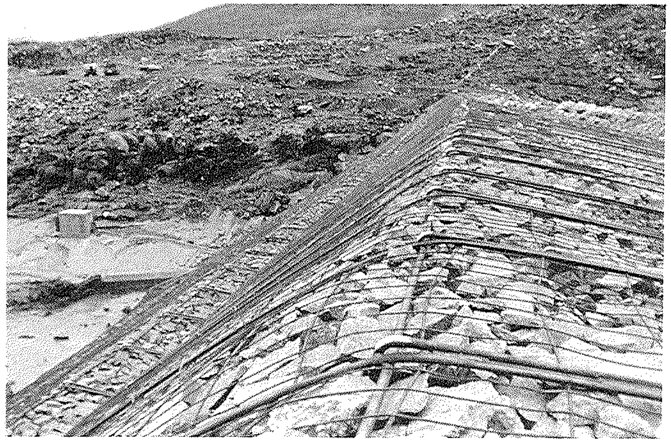


FIG. 8.—Reinforced Rockfill Prior to Flood

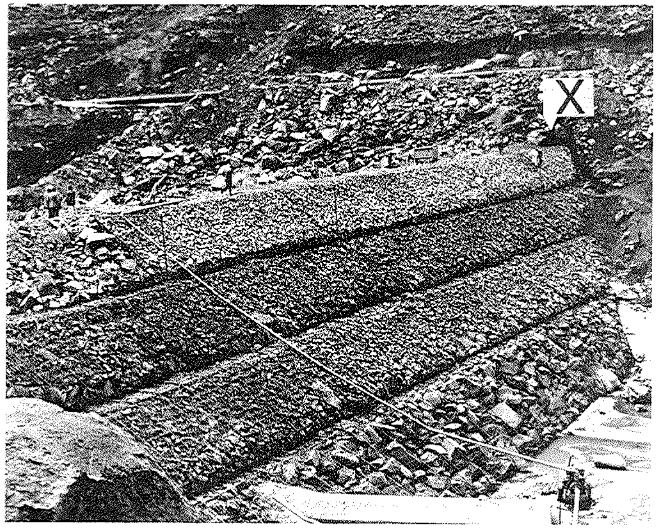


FIG. 9.—View of Left Flank During Placement of Fourth Layer—X Marks Residual Dolerite "Boulders" to which End Panels of Next Layer Were Fixed

that was virtually at the surface. However, on the left flank, above a height of about 20 ft (6 m) the panels under the end of each layer were placed on and doweled to what later were shown to be huge residual dolerite "boulders." The photograph in Fig. 9 shows the situation during the placement of the fourth layer and shows the boulders (marked with an X) on the left flank to which the end panels of the next layer were fixed. It can be seen that no protection was provided for the natural hillside material downstream of the face on the left flank. At the time of the flood a further three lifts of rockfill had been placed and the top of the rockfill was level with the high level outlet that can be seen on the left flank in Fig. 9.

Flood.—The only capacity for river diversion was provided by a 4-ft (1.22-m) diam, concrete encased, steel pipe. This outlet conduit requires some examination as it has a bearing on the failure. Originally a 5-ft (1.5-m) diam Armco pipe had been planned for river diversion but various problems, once construction had started, led to the adoption of the smaller steel pipe. While both the proposed Armco pipe and the actual 4-ft (1.2-m) diam steel pipe were quite capable of handling the river flow that could be expected for 90% of the year, there was one factor that had not been considered carefully enough. This was the time required to empty the reservoir that could form once the embankment reached a height greater than about 50 ft (15 m). The design flood for overtopping during construction had been taken as that having a 50 yr recurrence interval. This had a peak of 53,000 cu ft/sec (1,500 m^3/s) and an idealized hydrograph had been adopted for routing studies. The important point is that this routing was done assuming the reservoir to be empty prior to the arrival of the flood. The simulations indicated that overtopping, under design flood conditions, would not occur once the embankment reached a height of 80 ft (24 m). In fact the embankment was virtually this high when the flood occurred. However, the reservoir was almost full at the time and instead of the attenuation occuring as expected in the design studies the full flood passed over the rockfill. The reason for the reservoir being full was that for more than a month prior to the flood there had been minor spates in the river. Even though there was a respite of a week or so prior to the major flood the diversion conduit was too small to allow any significant drawdown of the reservoir.

The flood started on January 21, 1972 and had a peak of the same order as that assumed for the design flood. However, the flood volume was much greater than in the idealized hydrograph used in the routing studies.

Unfortunately, because all the site personnel lived in Queenstown, about 25 miles (40 km) from the site, and because the unsurfaced access road became almost impassable due to flooded creek crossings, only two people witnessed the full overtopping of the embankment. The first was the contractor's mechanic who succeeded in reaching the site and saw that full overtopping had developed and that the reinforced rockfill was showing no signs of distress. He then left the site. The second person was the local missionary who made his way to the site somewhat later in his Landrover and took some 8 mm cine film of the full flood pouring over the reinforced rockfill. By this time there had been full overtopping for about 12 hr and the reinforced rockfill was showing signs of distress on the left flank. Night fell and by next morning the embankment had been devasted, as shown in Fig. 10. About 75% of the rockfill and 65% of the compacted earthfill had been washed away.

The mechanism leading to the collapse became apparent when the cine films taken by the missionary were examined. At the time of overtopping the crest length along the downstream edge of the reinforced rockfill was about 560 ft (170 m) while at the toe of the dam the valley is about 130 ft (40 m) wide.

Thus under full overtopping there was considerable diagonal flow down both flanks. On the left flank this flow rapidly removed the loose rocky topsoil and then started eroding into the massive residual dolerite "boulder" formation. It appears that the huge "boulders" to which the left end of the fifth rockfill layer was fixed (marked with X in Fig. 9) were undercut on the downstream side and plucked out. Since the end mesh panels were fixed to these boulders with grouted dowels a hole was torn in the mesh. Once a large rip occurred in the face mesh, progressive failure could take place rapidly. Rockfill eroded out of the hole and the material in the higher layers collapsed downwards and was washed away until a slot had been formed to the surface. Flow then concentrated at the left flank and the slot was progressively deepened and

FIG. 10.—View from Left Flank after Flood

then widened towards the right flank. In fact it appears that most of the rockfill was washed somewhat diagonally from behind the still intact reinforcement out through the gap rapidly enlarging from the right flank.

Reconstruction.—The effect of the water sweeping through the breach in the rockfill was to scour the right flank down to fresh dolerite bedrock to an elevation of 50 ft (15 m) above the river bed. This scouring revealed quite clearly the depth at which one passed from residual dolerite "boulder" formation into what can be termed bedrock, something that was not easy to determine correctly from the borehole data.

Reconstruction was carried out using the same reinforcing system used originally. The big difference was that since the left flank had been scoured down to dolerite bedrock the ends of the layers could be satisfactorily constructed with there being no possibility of undermining.

The reason why no modifications to the reinforcement were deemed necessary was that until the undermining of the left end of the fifth layer occurred, the dam had taken 12 hr of overtopping without any signs of distress. Even after the failure the section of reinforced rockfill on the right flank was in near original condition and could be incorporated directly in the reconstructed embankment.

The dam was completed without any further overtopping and thus the "correctly constructed" reinforcement was not put to the test.

Post Mortem

It is always easy to be wise after the event, but it is certainly true that the factors that led to the failure at Xonxa merely involved common sense. Xonxa was designed and constructed in the light of what had happened at Bridle Drift and at Cethana. Every care was taken to prevent rocks falling onto or being washed onto the face, thus damaging the mesh. However, the main fault arose from the fact that, like all embankment dams, Xonxa was designed in terms of a two-dimensional slice. The problems associated with the ends of the layers of reinforced rockfill were thought of briefly and then left for the engineers on the site to resolve. Unfortunately it was only after the Xona failure that we heard of the experiences of Rhodesian engineers at the Lesapi dam. At Lesapi the embankment was only about 33 ft (10 m) high when overtopped. As at Xonxa the reinforced rockfill layers were not fixed to solid rock on the flanks and the only thing that prevented a major failure was the flood receeded before the flanks were completely undermined. Deep trenches had been carved on the flanks along the downstream toe.

This reprieve enabled the engineers to first fill the flood excavated trenches with lean concrete and then to continue construction by excavating and backfilling, with lean concrete, a trench immediately downstream of the rockfill toe. It is unfortunate that the Rhodesian delegate to the Montreal ICOLD Congress, who was to present the experiences at Lesapi, was for political reasons refused entry into Canada. Thus the rather obvious problem of the flanks was not forced under our noses during the design of Xonxa.

What then is the position with regard to flood control using reinforced rockfill? After Xonxa doubts were felt by many of the engineers concerned mainly because the final "correctly constructed" rockfill was not tested. It was reasonably certain that, provided loose rock could not be washed down the face and provided the flanks were protected, the system was satisfactory. However, there was no proof and the track record was not one to inspire confidence in a client. To a large extent the experiences in Australia at the Googong dam have provided the proof.

This is not the place for a detailed analysis of the Googong dam, recently constructed near Canberra, Australia. It is a dam in size and geometry very similar to Xonxa and the downstream rockfill was reinforced using the same approach as at Xonxa. There were three major differences. First, the downstream toe of the rockfill on both flanks was protected using a concrete filled trench, in much the same way as in the reconstruction at Lesapi. Second, the surface mesh was quite considerably heavier that that used at Xonxa. Thus a small break in the mesh would not easily "run" to cause a major hole as at Bridle Drift. Third, a substantial diversion tunnel was provided that could rapidly empty the reservoir.

In November, 1976 when at a height of about 35 ft (11 m) the Googong dam was overtopped by a major flood for a period of about 24 hr. The maximum depth of water over the rockfill was about 10 ft (3 m). The reinforced rockfill performed quite satisfactorily.

Conclusions

The technique of using reinforced rockfill to enable major construction floods to pass over a partly completed embankment dam can be successfully used to achieve large savings in the cost of construction of such a dam. However, the particular combination of weldmesh surface reinforcement coupled with long anchor bars, as covered in this paper, has the disadvantage that a relatively small break in the surface mesh can lead to a major failure. There is no second line of defense. Thus, utmost care must be taken to ensure that: (1) Damage to the surface mesh does not occur due to rocks being dropped onto the face during construction or washed down the face in a flood; and (2) undermining of the reinforcement cannot occur along the downstream toe down the valley sides. The weldmesh used on the downstream face should be considerably heavier than that necessary to prevent movement of the surface rockfill. A mesh consisting of 0.4-in. (10-mm) diam wires at 4-in. (100-mm) centers down the face and at 6-in. (150-mm) centers horizontally across the face is suggested.

It would appear from the experiences at the Xonxa and Googong dams that the anchorage force calculated using the modified Bishop slip circle equation is adequate. However, there does appear to be much scope for research into these anchors. Serious consideration should be given to the question of the time taken for significant seepage forces to develop in the rockfill. Use could be made of thin layers of cement stabilized sandy gravel on the surface of each compacted lift to considerably decrease the vertical permeability of the rockfill. If ingress of water from the filters was also prevented then significant seepage pressures would not develop during a construction flood. The deep anchor bars could then be dispensed with altogether. However, such a radical approach may not be economical or feasible in many cases and it suggested that attention be directed to using finite element techniques to achieve a more rational design procedure for deep anchorage bars. Such analyses will have to simulate the nonlinear stress strain behavior of rockfill, seepage body forces and slip of the anchor bars in the rockfill. There may be problems in such analyses in establishing when instability of the reinforced rockfill occurs, but nevertheless this approach should allow the design of a more rational distribution of the anchors.

Appendix.—References

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13586 REINFORCED ROCKFILL FLOOD CONTROL

KEY WORDS: Cofferdams; **Construction procedure; Dams (rockfill);** Design; **Diversion;** Erosion; **Failure; Flood control; Reinforcement;** Stability

ABSTRACT: The technique of using downstream zones of reinforced rockfill to enable the passage of major construction floods over a partly completed enbankment dam has great economic attraction. Three dams have been constructed in southern Africa using this approach, but their performance was less than satisfactory. The methods used in designing the rockfill reinforcement are outlined and the problems experienced at the three sites are covered with emphasis being placed on the major failure that occurred at the Xonxa dam. It is concluded that this construction technique can be employed successfully provided care is taken to prevent undermining of the downstream flanks and damage to the mesh by debris carried by the flood waters.

REFERENCE: Pells, Philip J.N., "Reinforced Rockfill for Construction Flood Control," *Journal of the Construction Division,* ASCE, Vol. 104, No. CO1, **Proc. Paper 13586**, March, 1978, pp. 85-95