A Partial Review of Barry Cooke and James Sherard 1987 Papers on Concrete Face Rockfill Dams (CFRD)

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Abstract. Two papers published by Barry Cooke and James Sherard in the Journal of Geotechnical Engineering v. 113:10 (1987) related to Concrete Face Rockfill Dams constitute the State of the Art on assessment, design, construction and performance of CFRD and continue to be a guide for the actual design and construction of such dams. An actual review of some items of those papers is presented, based on new data of the behavior of CFRD built within the 20 years that passed from 1987.
Key words: dams, rockfill, rockfill dams: design, construction, performance.

1. Introduction

The two papers written by Barry Cooke and James Sherard: Concrete Face Rockfill Dams I. Assessment and Concrete Face Rockfill Dam: II Design, published in the Journal of Geotechnical Engineering v. 113:10 (1987) of the American Association of Civil Engineers (Cooke & Sherard, 1987a and b) consolidated the basis for the design and construction of CFRD. A portuguese version of these papers was published by the Brazilian Committee of Dams by Oliveira (1987).

The two papers were based on the design, experience, and performance of the dams built at the time that amounted to 56 (above 50 m in height - Water Power Year Book, 2006). The Foz de Areia Dam (Brazil, 1980) 160 m the highest in the world, is repeatedly mentioned in the papers. Within these 20 years (1987-2007) the number of dams above 50 m built in the world jumped to 233. The actual highest is Shibuya - 232 m (China). However, the rupture of the concrete face of four large dams in 2006 has surprised the world specialists because such ruptures were not foreseen by consultants, designers and constructors. Figure 1 shows a photo of the Campos Novos Dam (202 m - Brazil).one the dams in which ruptures occurred in the concrete face.

These accidents led to a review of Barry Cooke and James Sherard basis for design and construction of CFRD. This review, as discussed below, represents only minor adjustments to the master work of those two great engineers, that had the courage to introduce and "reinforce" this alternative design in relation to the more traditional dams, aiming for economy, safety and speed in construction schedules.

The four dams that had the concrete face ruptured were repaired and are in operation with no risk of failure.

2. Review

The present review is limited to the performance of the rockfill, not only due to the limitation of pages and time for this conference, but mainly due to the limitations of the author's knowledge and experience in other aspects of the dam design.

The changes in the design of the joints, concrete face and reinforcements are referred only briefly. Some of such changes are still under discussion and escape the content of this paper.

2.1. Zoning designation

It is useful to use standard zoning designations and to adopt those common for the ECRD materials; Zone 1 for impervious, Zone 2 for the filter or transition zone directly under the concrete slab, and Zone 3 for the main rockfill. (Cooke & Sherard)

Figure 2 shows a CFRD cross section with the zone designation, and the table contains the materials and construction specifications. It would have been very useful to follow the proposed zoning designation to facilitate exchange of data and easy understanding.

A new zone 2A has been incorporated in new designs, that is, the extruded concrete curb introduced since the Itapebi dam by Resende & Materon (2000). This solution has been used under the concrete face and serves as a support for the slab and a form to the sand layer of zone 2. This detail will come later in this paper.

Figure 1 - Campos Novos Dam.

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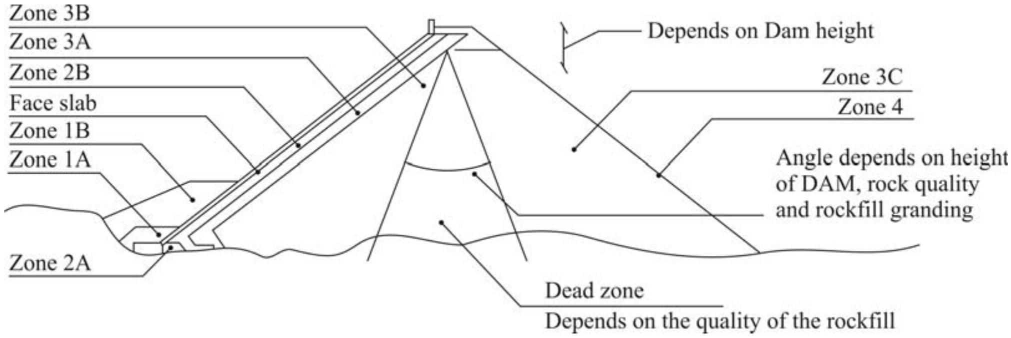


Material	Zone	Classification	Method of placement	Compaction
Imprevious	1A	Silty- Clay with rock fragments ≤ 19 mm	Compacted	0.25 m layer sheeptoot roller
Randon	1B	Silty- Clay with rock fragments ≤ 19 mm	Compacted	0.60 m layer 6 passes of construction equip.
Filter	2A	Sand and gravel $\phi \leq 5$ cm	Compacted	0.30 m layer 4 passes vibratory roller 10 t
Transition	2B	Gravel - $\phi \leq 100$ cm	Compacted	0.30 m layer 4 passes vibratory roller 10 t
Rockfill transition	3A	Massive basalt with ϕ_{max} 20 cm	Compacted	0.60 m layer 4 passer vibratory roller 10 t
Rockfill	3B	Massive basalt with maximum of 25% of breccias	Compacted	0.8 m layer 4 passes vibratory roller 10 t
Rockfill dead zone	3B 3C	Massive basalt with maximum of 40% of breccias	Compacted	1.2 m layer 4 passes vibratory roller 10 t
Rockfill	3C	Massive basalt with maximum of 25% to 40% of breccias	Compacted	2.0 m layer, 4 passer vibratory rolle 10 t
Downstream	4	Large blocks of massive basalt or basaltic breccias	Placed	

Figure 2 - Concrete face Rockfill Dam typical cross. Section and specification.

Looking into many projects of CFRD this author has seen that the zones suggested by Cooke & Sherard follow the proposed sequence, but the zone designation unfortunately are named with different numbers and notations.

2.1.1. The rockfill behaves as a whole

> *The rockfill embankment is in three zones of increasing layer thickness to give a desirable transition of compressibility and permeability from upstream to downstream. Lowest compressibility is desirable in the portion of the upstream shell which transmits water load to the foundation. Because most of the water load passes into the foundation through the upstream shell it is desirable that the compressibility of zone 3B be made as low as practical to minimize slab settlement. The downstream zone 3C takes negligible water load, and its compressibility has little influence on the settlement of the face slab.* (Cooke & Sherard)

Considering the materials and compaction specifications in the table of Fig. 2 it is clear that the design of this hypothetical dam, follows Cooke's and Sherard's recommendations as stated above. However, when one looks more carefully in the behavior of CFRD one can see that the rock fill behaves more as a whole than as independent zones 3B and 3C. As an example, Fig. 3 shows the end of construction settlements measured in the Itapebi dam. There is a shift of the settlements to zone 3C. During impounding the water load will reach zone 3C.

A simple analysis of the stresses that develop in a CFRD is shown in Fig. 4 at the end of construction and after the reservoir filling for points within the embankment. It is clear that in the upstream shell there is a change in the direction of the stresses from the end of construction to the full water load. Near the face the shear stress decreases and then increases again. In the downstream shell the change in direction of the stresses is practically negligible and the stresses increase with the water load.

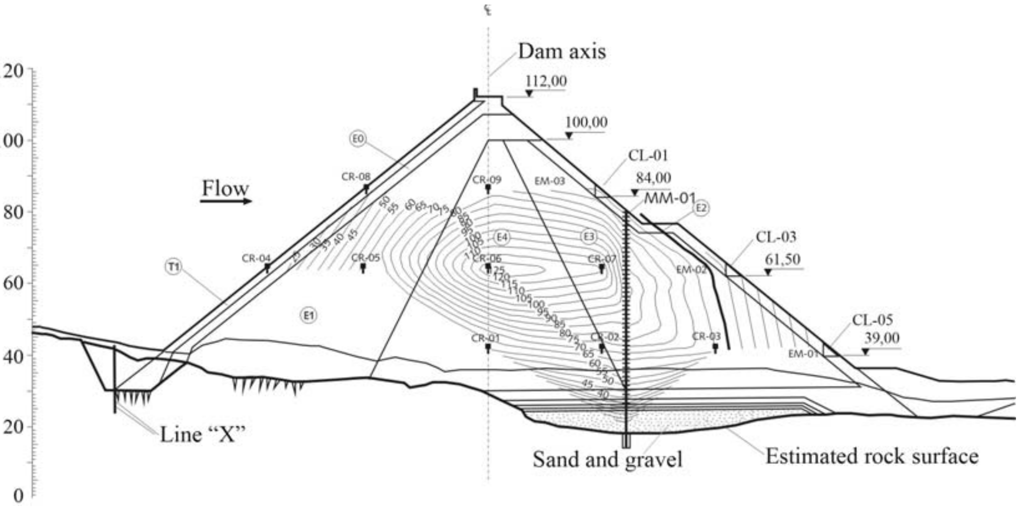


Figure 3 - End of construction settlements measured in the Itapebi Dam (Albertoni *et al.*, 2001).

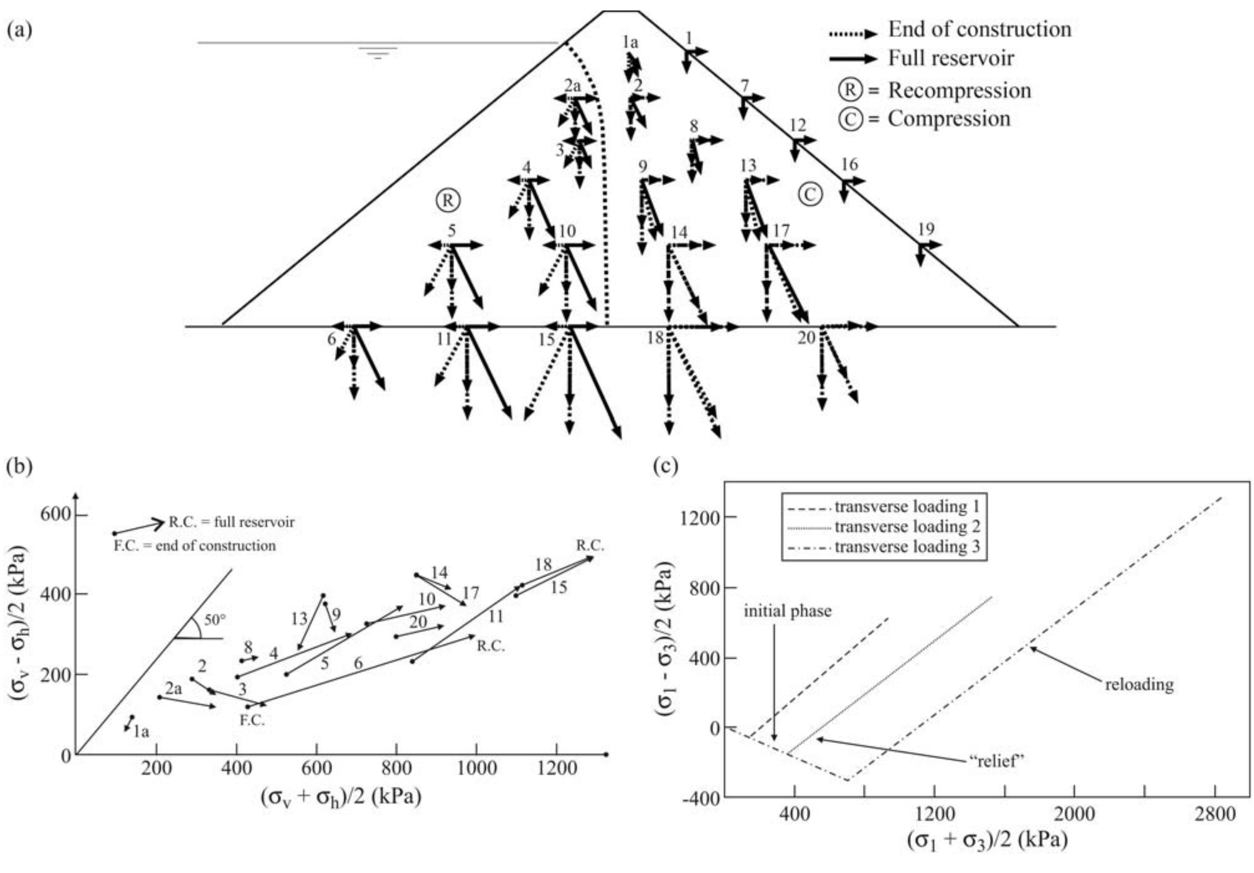


Figure 4 - Stress analysis of a CFRF dam. (a) Vertical and horizontal stresses during construction and full reservoir (Oliveira, 2002). (b) Horizontal and vertical stress paths at end of construction and full reservoir (Oliveira, 2002). (c) Stress paths for laboratory tests (Basso, 2007).

It is possible to separate zones of reloading in the upstream shell and zones of continuous loading downstream. The compression modulus will be different in the zones, as well as the displacements, as shown in the example in Fig. 5.

When the upstream zone is made of gravel and the downstream of rockfill, the lower compressibility of the gravel can lead to bending in the upper third of the face slab as happened in Aguamilpa Dam (Fig. 6). For higher dams, the specifications for compaction in the downstream shell

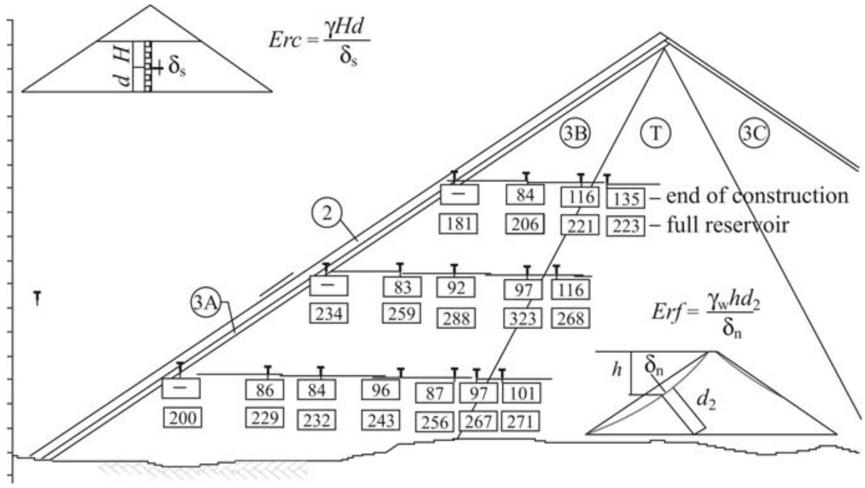
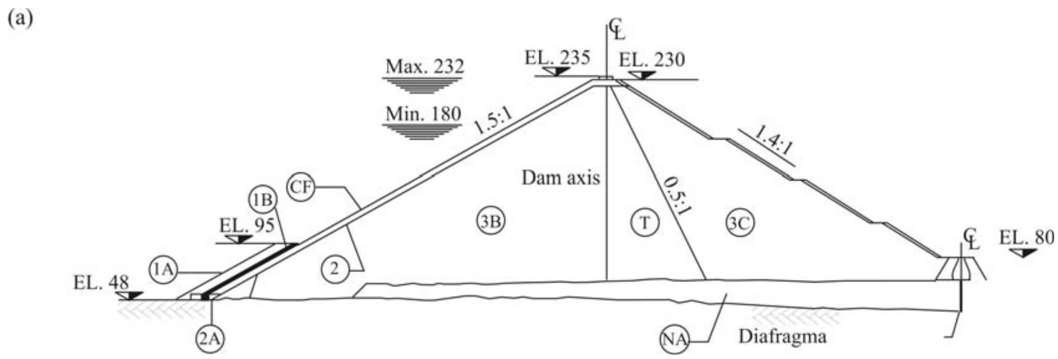


Figure 5 - Deformability modulus at end of construction and full reservoir - El Cajon Dam (Sandoval *et al*., 2007).



Specification	Zone	Compaction procedure	Compaction specification
Random	1A	Not compacted, just placed 80 cm	
Fine silty sand, ϕ 0.2 cm	1B	Not compacted, just placed 30 cm	
Alluvial gravel and silty sand mixture, ϕ 3.8 cm	2F	Compacted layer 30 cm	Layers 4 passes of 100 kN SDVR
Crushed alluvial gravel sand mixture, ϕ 7.6 cm	2	Compacted layer 30 cm	Layers: 4 passes of 100 kN SDVR Face: 6 passes of 40 kN SDVR or 130 kN PC
Dredged alluvial, ϕ ≤ 4.0 cm	3B	Compacted layer 60 cm	4 passes of 100 kN SDVR
Rockfill 3C with reduced ϕ max 5.0 cm	T	Compacted layer 60 cm	4 passes of 100 kN SDVR
Rockfill (ignimbrite), ϕ 100 cm	3C	Compacted layer 120 cm	4 passes of 100 kN SDVR
Concrete face	CF		
Natural alluvium	NA		

Figure 6a - Aguamilpa Dam.

(zone 3C) have been revised to make then less compressible.

2.1.2. The leakage through the rockfill

The increasing permeability from zone 2 progressively through zones 3A, 3B, and 3C (Fig. 2) is desirable during construction, in the event of a flood before the concrete face is placed. After the concrete face is placed there is no credible face problem that could cause more leakage than the rockfill could handle without damage. (Cooke & Sherard)

In another part of the first paper, Cooke & Sherard discusses the advantages of a placement that will lead to a stratified rockfill.

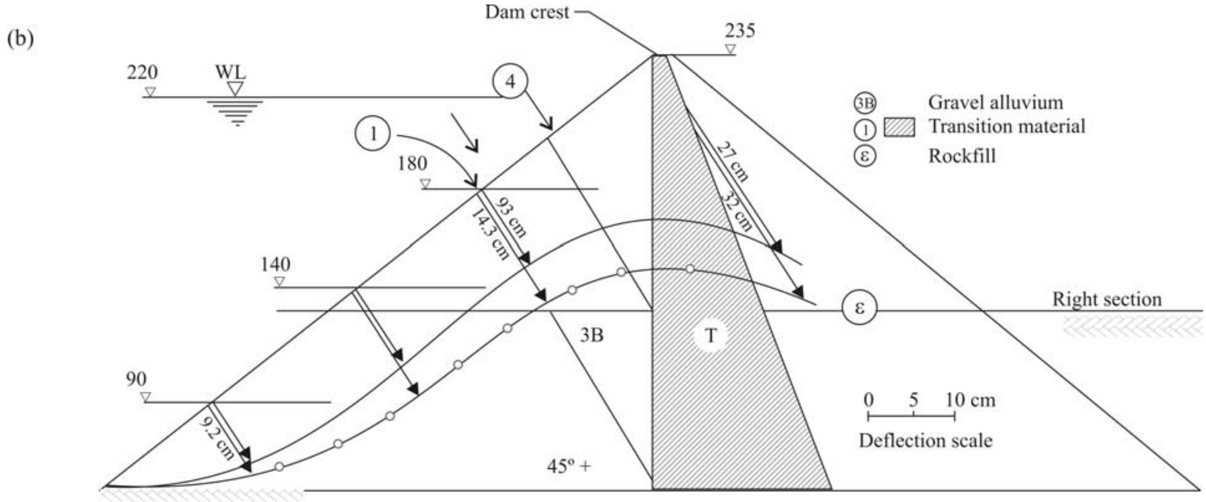


Figure 6b - Face displacements of Aguamilpa Dam (Mori, 1999).

There are no technical disadvantages to the preferred method of placement in segregated layers. In addition to lower cost, there are several advantages. The stratification assures that any flow through the rockfill embankment will travel much more easily in the horizontal direction than vertically. (Cooke & Sherard)

The leakage that passes through fissures, cracks and even fractures at the face of CFRD are controlled by the sand layer, and are considerably less than the flow that will start a progressive removal of the downstream rock blocks. Leakages as high as 1000 to 3000 l/s have been measured in the CFRD Barra Grande, Campos Novos (2007) without any signs of trouble in the downstream rock slope, confirming Cooke & Sherard position. However, if a flood reaches the dam face, before the placement of the concrete slab, the anisotropy of the permeability will not be favorable, because it raises the phreatic line and concentrates the flow in the bottom of the segregated layers as demonstrated by Pinto (1999), in a laboratory experiment with sand and gravel. Under a theoretical point of view Cruz (2005) demonstrated that the higher the point in which the phreatic line reach the downstream shell, the worse is the condition of instability.

2.2. Construction

Regarding construction stages, Cooke & Sherard are quite liberal, as can be seen as follows.

There has been some thinking that the rockfill dam should be completed to full height before starting the placement of the concrete face. The construction and performance experience at the Areia, Salvajina, and Khao Laem Dams conclusively shows that the face slabs can be placed in any sequence convenient to the contractor to obtain maximum schedule and cost benefits.

At the Areia Dam - the highest CFRD to date (160 m), and with the highest settlement - the concrete slab was placed on the lower 80 m of the dam height before the rest of the embankment was completed. The top of the first-stage face slab in the center of the valley moved downstream normal to the slope about 0.6 m while the rest of the embankment was being completed, causing no problem. (Cooke & Sherard)

The movements or displacements of a rockfill occur in two steps or times: a fast initial movement that follows the load increment, ie, the raising of the rock fill or the water in the reservoir. And a second step due to the progressive accommodation of the rock blocks and its progressive breakage due to the reorientation of the stresses. In narrow valleys, according to Cooke & Sherard, *An arching effect develops during the construction of the rockfill that progressively relaxes with time*. This second step is called creep. Usually it may represent a small fraction of the first step, but it is continuous and its effect are felt mainly in the upper half or third of the dam face deflection.

In Campos Novos dam (203 m - Brazil) the face slab was built in 3 phases. The first between March 2003 and August 2003, the second during September 2004 and March 2005 and the final stretch (mostly above the max. water level) at the end of construction. Figures 7 and 8 shows that the vertical displacements measured by settlements cells, due to the construction loads, were still under progress when the first stage of the slab was built. Figure 8 shows the corresponding horizontal displacements. Figure 9 show the face slab movements of the Xingó CFRD during 5.5 years of monitoring.

2.3. Construction materials

Apart from the somewhat rigid requirements for the sandy layer (zone 2) and the transition material (zone 3A) Cooke & Sherard are quite liberal in accepting rocks for the rockfill with relatively low resistance and with fines.

The most important properties of the CFRD embankment are low compressibility and high shear strength.

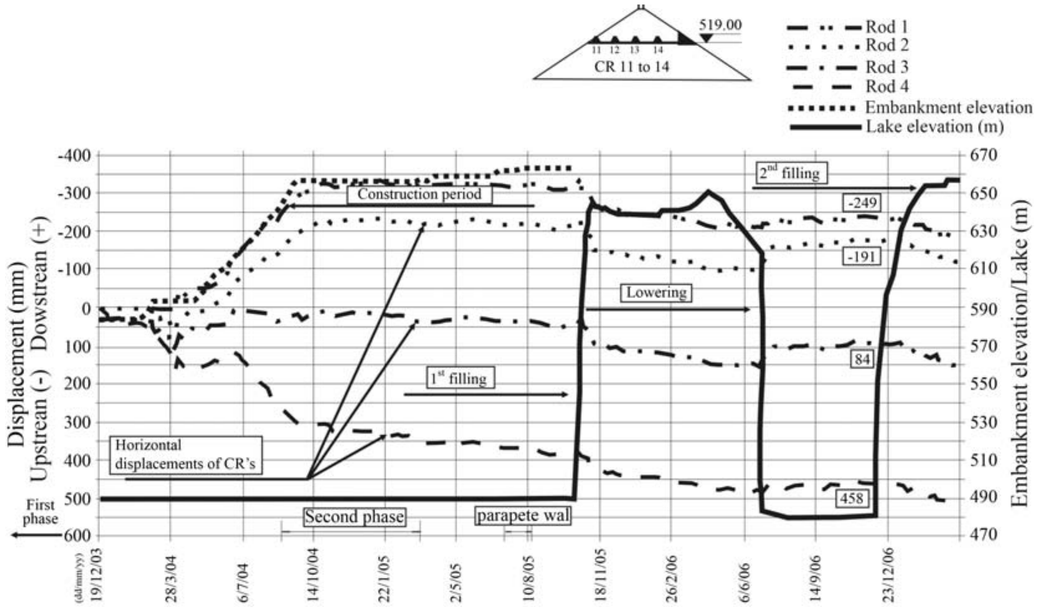


Figure 7 - Horizontal displacements of 1 to 4 CR 15 to 18 (Cruz, 2007).

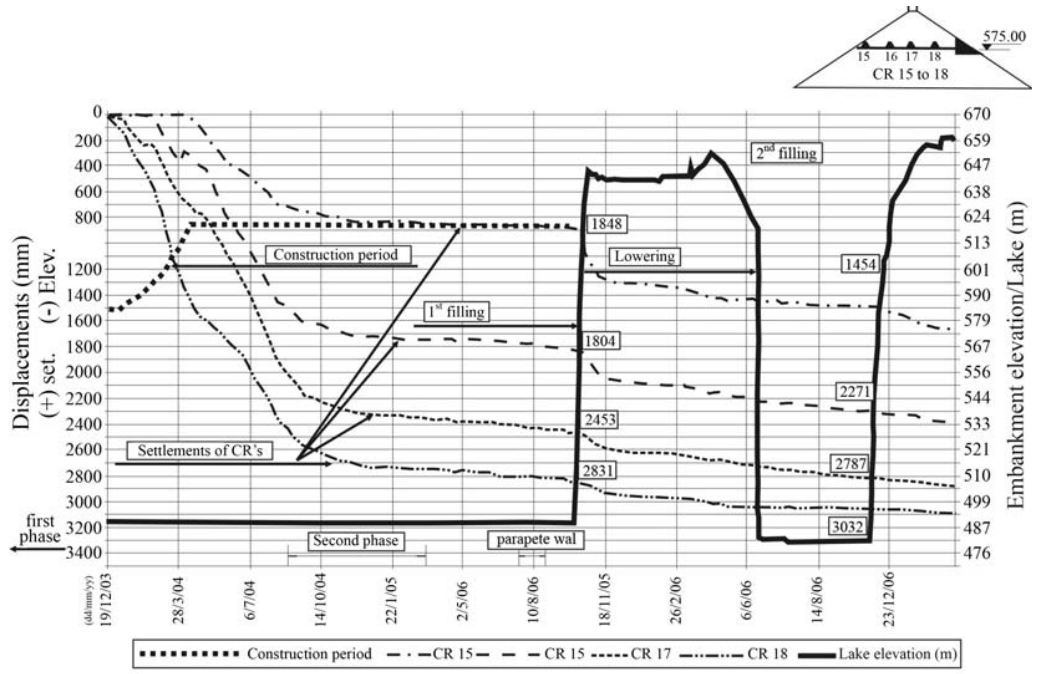


Figure 8 - Vertical displacements of 15 to 18 - stake 13+10,00 - El 575 (Cruz, 2007).

Usually rockfill is highly pervious, but less pervious rockfill can be used in CFRDs by providing special interior drainage zones. As a general rule any quarried hard rock with an average particle size distribution having 20% or less finer than the n. 4 sieve, and 10% or less finer then the n. 200 sieve, will have the needed rockfill high shear strength and low compressibility. These limits may be a better means of defining rockfill by sizes than the common specification requiring a maximum percentage of particles smaller than 2.5 cm.

When a rock fill contains a fine content exceeding these limits, commonly the final evaluation of suitability can be made on the basis of the trafficability of the rockfill surface when the material is thoroughly wetted.

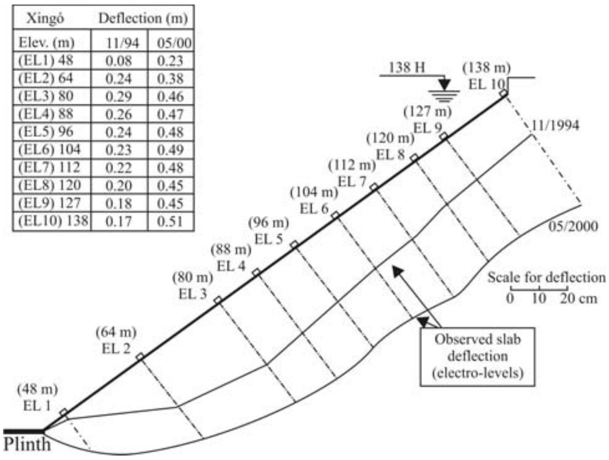


Figure 9 - Face slab movements of the Xingó Dam (Penman & Rocha, 2000).

A stable construction surface under travel of heavy trucks demonstrates that the wheel loads are being carried by a rockfill skeleton. An unstable construction surface, with springing, rutting, and difficult truck travel, shows that the volume of soil-like fines is sufficient to make the rockfill relatively impervious. Where the surface is unstable, the fines dominate the behavior and the resulting embankment may not have the properties desired for a pervious rockfill zone. (Cooke & Sherard)

Restrictions regarding the materials are related to problems in compaction and low permeability, when a drainage filter is provided to drain the leakage. In fact, an analysis of any table containing data of the rockfill materials used in CFRD shows that practically every rock has been used in CFRD rockfill with restrictions only to the presence of fines that would control the behavior of the rockfill. A possible restriction to be observed refers to the use of two very different materials regarding compressibility because, as mentioned before, the dam behaves as a whole. Any more compressible zone will concentrate the displacements and affect the overall distribution of the stresses within the rockfill.

A simple exercise can be done. Let's consider a CFRD 120 m high, with outer slopes of 1,3 (h):1,0 (v), as shown in Fig. 10. The displacement at any point in the slab can be computed by the expression $\Delta = \sigma l/E$, being σ the water pressure, l the distance from the slab to the foundation and E the modulus of compressibility. If E is taken as constant the displacements are those shown by the line 1st solution. If one considers that in the first half the E is the E_R for reloading (as discussed) and in the second half E is E_C (loading) and that $E_R \cong 2E_C$ (as observed) the displacements of the slab are those in the 2nd solution. This simple procedure leads to slab movements very similar to those measured in CFRD (see Fig. 9). If E_C is taken as E_R divided by 3 the displacements correspond to the 3rd solution (arrows in Fig. 10).

2.4. Zone 2

The early and primary purpose of the thin zone of finer rock directly under the slab was to provide uniform and firm support for the concrete slab. Crusher-run minus -1,5 - 7.5 cm rockfill has been used.

Recently there has been a trend toward making zone 2 grading have a sufficient quantity of sand-sized particles and fines to improve workability, reduce excess concrete and have reliably low permeability, and have an approximate filter grading.

For face compaction, the roller is first pulled up the slope without vibration for several passes, and then given four upward passes with vibration. A new and promising development is the use of a backhoe-mounted plate vibrator. On several new dams, the plate vibrator is being specified for zone 2 compaction adjacent to the toe slab, and optional for face compaction.

At rainy sites, it is desirable to place the erosion protection as soon as possible after embankment placement. Surface protections comprised of about 50-76 mm of shotcrete or a sprayed sand asphalt skin have been equally satisfactory for general erosion protec-

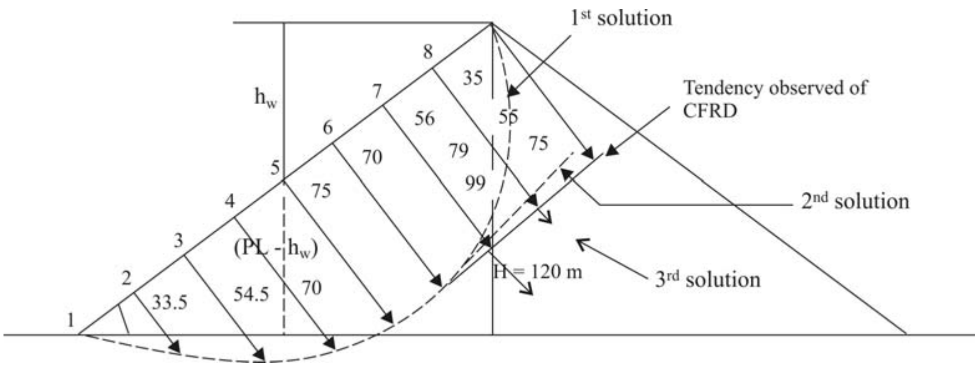


Figure 10 - Slab displacements.

tion, but cannot be relied upon alone to resist large concentrations of runoff from the abutment. (Cooke & Sherard)

Cooke & Sherard recommendations regarding zone 2 have more details than the 3 paragraphs above and should be implemented.

An alternative for the support of the concrete slab was introduced by CNO - Contractor Norberto Odebrecht S.A. for the Itapebi and Itá CFRDs (Resende & Materón, 2000). An extruded concrete wall built as support for the concrete face replaces all the troublesome operations of compaction along the slope of zone 2 and eliminates the asphalt emulsion spray. It serves also as a support to the compaction of the sand layer and provides protection of the sand against rain erosion. Figures 11 and 12 show the curb machine and curb details.

2.5. Joints and reinforcing

The perimeter joint always opens and offsets moderately when the reservoir is filled, and is a potential source of leakage if not well designed, inspected, and constructed. For dams of low to moderate height (less than about 75 m), the joint movement has commonly been only a few millimeters, and joints with current water stop details have usually remained watertight. For some of the higher dams, the joint openings and displacements have been several centimeters. At the 160 m high Areia Dam, the opening in one area was 2.5 cm and the offset 5 cm. No joint leakage occurred, but is probable that the central bulb waters top was ruptured.

Today longitudinal reinforcing is continued through the joints without waters tops. This is considered good practice, is more economical, and has been adapted for more recent dams. With the longitudinal steel passing through the construction joint, there is no need to use water stops.

The face slab has usually been placed in 12 - 18 m wide strips, 15 m being must commonly used. The dimensions should be left to the contractor. (Cooke & Sherard)

In the practice today the horizontal joints are mainly construction joints. The vertical joints in the tension zone near the perimeter joint and near the abutments have special design details (Perez *et al.*, 2007) but in the compression zone the joints usually are built side by side, with a water stop in the bottom. Regarding the perimeter joint ant the vertical joints in the tension zone the concern has been with leakage and only few changes have been introduced since Cooke & Sherard papers of 1987.

The larger displacements that have damage the CFRD with dumped rockfill would not occur in the compacted rockfill. The compressive stresses that develop in the compression joints or within the slaps, due to the movements of the rockfill, would be taken by the compressive strength of the concrete slab. These assumptions were made in the design of CFRD until the recent accidents in four high CFRD. To deal with this problem in high dams in narrow valleys, some of the vertical joints in the compression zones have been left "open", *i.e.* with some space between them to allow movements and avoid the concentration of stresses (Fig. 13). Similar solutions were incorporated in the Shibuya Dam and in the repairs of other dams.

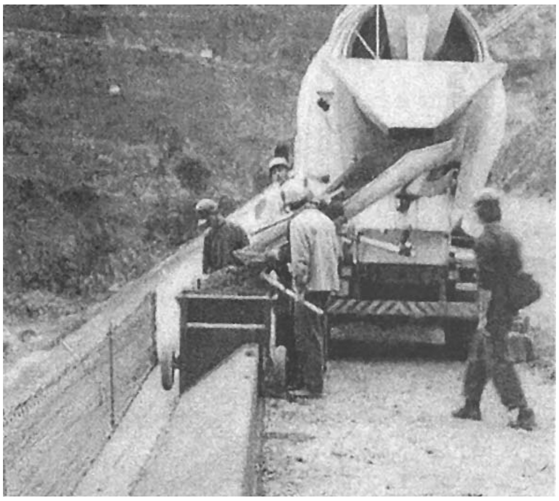


Figure 12 - Curb machine.

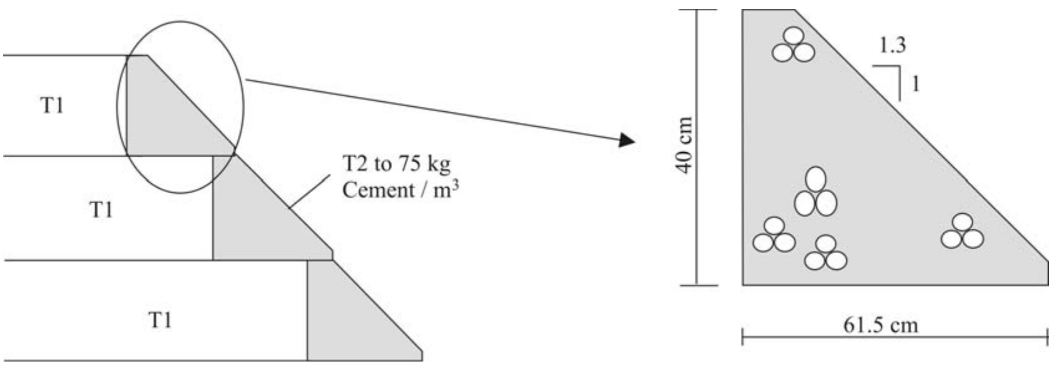


Figure 11 - Curb detail.

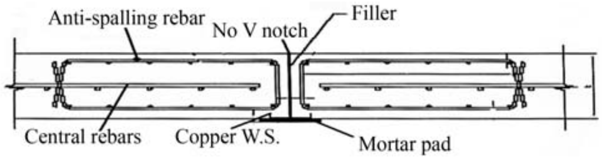


Figure 13 - Karanjulca face slab joint at central compression zone (Perez *et al.*, 2007).

2.6. Instrumentation

The empirical design that prevails in CFRD for the last 50 years has suffer improvements and modifications, because these CFRD have been well instrumented and through the observed data one can analyze the behavior of these CFRD. The views and considerations of Cooke & Sherard of 1987, are actual as can be seen in the following two paragraphs:

> *Instrumentation on CFRD dams has been important in gaining knowledge that has led to design, improvements in joint and face-slab design, evaluation of rock and rockfill, and zoning in the rockfill. The results have given confidence in proceeding with higher dams. Instrumentation is not a requirement for safety monitoring. However, a minimum amount is used.*
>
> *Areia Dam would not be a CFRD without the pioneering engineering of the Hydro-Electric Commission of Tasmania, and the publication of instrumentation results. The owners of the Areia (160 m) and Salvajina (148 m) Dams have extended the practice. The design of the CFRD is essentially empirical. It is based on experience and judgment. Instrumentation results are a major factor in the words "empirical", "experience", and "judgment". Instrumentation data on existing and new dams are important to continuing progress.* (Cooke & Sherard)

But the recent accidents in Mohale, Barra Grande, Campos Novos and Tianshenqiao 1, have shown that the intricate mechanism of stresses transmitted from the rockfill to zone 2, the extruded concrete and the concrete slab are not yet well understood, and that the instrumentation existing in those dams were not able to foreseen what happened.

The movements of the rockfill towards the valley that result in the intricate mechanism mentioned above are not measured. One measures settlements and displacements towards upstream or downstream, but the complete movement is needed. The bench marks placed on the crest and the downstream slope of the dam provide this information, but only in the external contour of the dam. A new instrument is necessary to provide the complete displacements of the dam body. Mathematical models have been developed, but one can not compare predictions with real displacements. Attempts to measure the stresses in the slab with strain gages were done in the Mohale Dam, but in this author's opinion view one still do not have a complete picture of the stresses in the concrete slab.

3. Conclusions

In the 20 years that passed since 1987, more then 177 CFRD above 50 m have been built around the world, most of which have had Dr. Barry Cooke as a consultant. The two of 1987 discussed above papers have guided the design and construction of must of these dams. The purpose of this paper is solely to bring some aspects of CFRD behavior into a discussion that has already been happening in the last 20 years.

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