THE EVOLUTION OF THE CFRD DAM

ALBERTO MARULANDA
TREND IN HEIGHT OF CFRD DAMS WITH TIME

(Cooke, 1997, extended to 2006)

- **Dumped Rockfill**  ○  **Compacted Rockfill**

1. Strawberry Creek  
2. Salt Springs  
3. Paradela  
4. Quioch  
5. New Exchequer  
6. Cethana  
7. Ananchey  
8. Areia  
9. Khao Laem  
10. Segredo  
11. Aguamilpa  
12. Yacambu  
13. Tianshenqiao  
14. Campos Novos  
15. Barra Grande  
16. Cajon  
17. Mohale

A. 68 CFRDs completed between 1990 and 2006, height 40 to 120 m
For this dam, that was the tallest in the world for 23 years, vertical joints with openings of one inch were included and horizontal joints were built as construction joints.
Eight of the central vertical joints closed completely, and in three the concrete failed by crushing. Some of the horizontal joints also experienced concrete crushing. At the moment, it was believed that the major source of leakages were the cracks near the union with the abutment, where no perimeter joint was built. The cracks in the central compression zone have similarities with the ones observed in the recent incidents of the Brazilian dams of Barra Grande and Campos Novos and in the Mohale dam in Lesotho where severe cracking occurred during the first filling of the reservoir. Leakage in Salt Springs reached 450 l/s. The empirical solution adopted after Salt Springs consisted in increasing the number of joints and in introducing compressible materials, as done in Bear Creek [1].
Face slab deflections after 2 years (1933)

Face slab deflections after 27 years (1958)

Joint opening 27 cm

Joint closure 40 cm

Joint opening 13 cm

Open joints

Cracks

Crushed concrete
SALT SPRINGS DAM
The empirical solution adopted after Salt Springs consisted in increasing the number of joints and in introducing compressible materials, as done in Bear Creek. In addition, a perimeter joint and a hinges Slab parallel to the canyon were included to decrease the demands on the slab near the abutments. In the Bear Creek dam, leakage was reduced considerably (112 l/s) and also the cracking was limited to some cases were superficial spalling occurred in the walls of the joint.
The Paradela Dam (120 m) is the tallest structure built with the technology known until the 1950s. In reference [2] the behavior of the dam and the concepts considered for its design are presented. The design was absolutely empirical and based on additional elements that included the creation of new perimeter joints (two perimeter slabs were built, like the ones used in Bear Dam), opening of vertical joints with compressible elements (7 cm) and of 3 cm in the horizontal joints.
During the repair after the first reservoir impounding, joint openings were further increased by cutting the border of the joints to widen the space for movements trying to relief the compression stresses in the concrete face. The materials used in the fillings were so compressible that, even before the reservoir impounding, it was observed that the joints between the slabs were closing as a result of the deformations in the filling by their own weight. With the reservoir load the deformations reached a value of 2.04 m in the direction normal to the face.
Excessive stresses generated by movements in the first and second impounding produced some cracks in the borders of the slabs. However, the largest cracks were reported in the perimeter slabs. The leakage reported in the first years of operation exceeded 3 m³/s; confirming the capacity of the fill to manage high levels of leakage without the risk of failure of the structure.
PARADELA DAM

MAIN REPAIRS AND JOINTS REOPNEII

LEGEND
- JOINTS REOPENED IN 1958
- JOINTS REOPENED IN 1959
- SLABS RECONSTRUCTED IN 1959
- REPAIRS OF RELATIVE IMPORTANCE IN 1959
- NEW FOLD OF THE COPPER SEAL IN 1958
- SLABS WITH CRACKS OR VERY LOCAL DEFECTS REPAIRED
- AREA MORTAR GRouted UNDER SLABS 1959
- AREA MORTAR GRouted UNDER SLABS 1958 AND 1959
- JOINTS WITH LOCAL POURING DEFECTS REPAIRED IN 1958

NOTE:
- THIS IS A TRUE VIEW OF THE CONCRETE FACE

MOVEMENT OF PERIMETRAL SLABS
LESSONS LEARNED FROM THE FIRST GENERATION OF CFRDS

• The relevant lessons from the first generation of these dams referred to the effects that the excessive movements of the fill generated on the face slabs. The deformations were associated with the low deformation modules that were obtained from the placement process. The experience in these dams seemed to indicate that in the evolutionary process of trying to eliminate the cracks, by compression, with the creation of more deformable joints, the cracks were reduced but the leakage increased as a result of the greater number of joints and the opening of those that did not closed. This is a very relevant assessment for modern dams that will be further discuss later in this paper.
• After the construction of the Foz de Areia Dam the construction of several concrete face dams with heights below 130 m and rockfills basically composed of basalts started in Brazil. The process of placement and compaction was relatively homogenous in these dams with materials lifts in the dam of 1 m in the upstream shell and 2 m for downstream shell. All of these dams were considered successful experiences, even though, leakage between 300 and 500 l/s were recorded. Some of the dams, like in the Xingo case, experienced cracking in specific places that illustrated the relevance of precluding sudden changes in the deformation pattern of the slab, avoiding hard points behind the face (Marulanda and Pinto).
• Between 1975 and 1990, central core rockfill dams of great heights and in very narrow canyons were built in Mexico (Chicoasen dam, 260m) and in Colombia (Chivor 238m and Guavio 248m dam).
GUAVIO DAM (248m)
CHIVOR DAM (238M)
• The behavior of these Colombian and Mexican dams demonstrated the fundamental concepts of the rockfill behavior postulated since the 70’s by Marsal [3]. The strength of a rockfill depends on the hardness of its particles, but its compressibility depends much more on gradation and compaction process. Materials with particles of less strength could have less compressibility if they are properly gradated and compacted. The process of grain breakage substantially decreases by adding water during compaction and by the presence of good gradation where the finest materials fill the gaps between grains.
Aguamilpa Dam that for more than 15 years was the tallest dam (190m) of its type in the world. Even though the shell upstream of the dam was built with gravels of high deformation modulus, the greater compressibility of the rockfill used in the downstream shell generated an unusual situation by introducing a non-uniform pattern of deformations in the upper part of the face.
AGUAMILPA DAM
Maximum Section and Material Gradations
AGUAMILPA DAM
Normal Face Displacements
AGUAMILPA DAM
Cracks in the Concrete Face
Settlement contours at Aguamilpa Dam as of October 2011
(First filling is included)
Towards the final years of the last century there was a clear tendency to consider that the concrete face dams could be viable for heights of more than 200 m, basically without major changes in the configuration and design procedures commonly used. In some cases it was considered the ideal dam, and therefore, to some people there was no limit to its height.
Sherard & Cooke 1985 CFRD ASCE Symposium:

“The CFRD is an appropriate type in the future for the very highest dams. For a 300m high CFRD constructed of most all rock types, acceptable performance can be predicted, based on reasonable extrapolation of measurements on existing dams”

Sherard & Cooke 1985 CFRD ASCE Symposium:

“For CFRD with compacted rockfill and a compacted upstream face, the thickness increment was decreased to 0.003H, and even to 0.002H or less. These slabs have given satisfactory performance, an there is a current general trend toward thinner slabs.”
Sherard, 1985 CFRD ASCE Symposium:

“....The writer believes that it is likely that the not distance future evolution of the CFRD could arrive at a constant slab thickness of the order of 8 to 10 inches, even for high dams, with simpler and more economical joint seals.”

Cooke 2000 Beijing Symposium:

“There has since been no experience to change that conclusion. There have been leakage incidents, and for the CFRD “acceptable performance” can include a leakage incident.”

“Experience with existing dams has not identified areas in design which require significant change in design practice for the next generation of higher dams, 190-230m”
Towards the end of the last century, a new stage in the evolution of CFRD dams started. Considering that the height limitations for a dam of this type were overcame, the more or less simultaneous construction of very high dams started in China (TQ1), Brasil (Barra Grande and Campos Novos) and Lesotho (Mohale).
GENERAL TRENDS OF HIGH CFRD’s in 1998-2005

- About 190 m high
- Without anti-spalling reinforcement along compression joints
- Thickness reduction of the concrete slab
- In low-seismicity areas, outer slopes increased to 1.3 to 1 and 1.25 to 1.
- Use of an extruded concrete curb as surface protection before placing the slab
- Aguamilpa, a gravel fill dam, is not an adequate precedent for the behavior of high rock fill dams. Settlements were very low due to high modulus of fill
CAMPOS NOVOS DAM
CAMPOS NOVOS: GENERAL CONCRETE FAILURE MAPPING

Construction joint

568,00
Campos Novos CFRD, Brasil, 2006
JOINT 16 - 17

CONCRETE RAISED UP -26 CM

ZONE CLOSE TO THE INCLINED CRACK IN SLAB 17
Inclined cracks in slabs 19, 20, 21, 22
Mohale dam embankment: face slab
Incident at the Mohale Dam (2006)
Shear failure along joint between Slabs 17/18
Location of cracks and failed joint
• When the three serious incidents of the dams in Brasil and Lesoto took place, two very high dams were being built, one in Mexico (Cajón, 188m) and one in Island (Karajnukar, 200 m). The rockfill materials used in these dams were different from the ones used in the problematic cases.
• Concept of obtaining required modulus between certain limits and with good gradation depending on compaction effort. Low strength particles can still produce and acceptable rockfill if well graded.

• Modulus between 100MPa and 50 MPa can be obtained even with weak particles. Compaction effort in terms of number of passes, thickness of layers and weight of vibrating roller. Water sluicing is also a must.

• Obtaining good gradation with hard rocks is very difficult and expensive. In basalts is even more difficult if they are columnar basalts.
• Most difficult aspect to evaluate is the effect of compaction in the materials characteristics. How much does it degrade.

• Fill test is advisable. Degradation is easily measured. Compaction parameters can be evaluated using odometer and plate load tests not to obtain final modulus but differences among them.

• Fill design should be optimized based on available materials. Hauling distances should be optimized. This is standard practice even for dams in narrow canyons where variable rock conditions are expected in quarries.
CAJON CFRD: MEXICO
CAJON DAM. FACE JOINT MEASUREMENTS
FIRST FILLING OF THE RESERVOIR (EL. 309.2)
CAJON CFRD: SLUICING OF ROCKFILL
## ROCKFILL: MATERIAL PROPERTIES

<table>
<thead>
<tr>
<th></th>
<th>Compresive Strenthen</th>
<th>Weight</th>
<th>Gs</th>
<th>Absorption</th>
<th>LA Abrasion</th>
</tr>
</thead>
</table>
|                | Mean [Mpa] | Max [g/cm³] | Min | [Mpa] | Mean [%] | Max | Min | Mean [%] | Max | Min |%
| Shallow Rock   | Dry       | 70.5   | 97.6 | 46 | 23.6 | 4.42 | 4.31 | 4.54 |
|                | Saturated | 52.9   | 73.2 | 34.5 |      |      |      |      |
| Underground Rock| Dry       | 124.7  | 131.89 | 111.05 | 23.3 | 4.42 | 4.31 | 4.54 |
|                | Saturated | 104.7  | 110.8 | 93.3 |      |      |      |      |
| Specification  | Barra Grande | 119.2 | 165.7 | 60.8 | 2.84 | 0.76 | 1.91 | 0.2 |
|                | Vesiculo Amigdaloidal | 99.4 | 189 | 16 | 2.6 | 2.2 | 4.8 | 0.3 |
|                | Dense Basalt | 119.2 | 165.7 | 60.8 | 2.84 | 0.76 | 1.91 | 0.2 |
|                | Saturated | 101.9  | 204.2 | 35.4 |      |      |      |      |
Settlement contours measured at El Cajón Dam as of May 2011
(First filling is included)
CAJON DAM. FIRST FILLING OF THE RESERVOIR
FACE DEFLECTION @ CENTER SLAB

- Cajón 29/08/2006. 125.2m
- Cajón 12/09/2006. 136.4 m
- Campos Novos 19/10/2005. 129.4m
- Campos Novos 20/10/2005. 135.2m

Height [m]

Normal Face Deflection [mm]
ANALYSIS REQUIREMENTS

• Development of a three dimensional model
• Construction sequence
• Modeling the structural elements
• Constitutive models for geomaterials
• Incorporation of interface behavior between different elements of the structure
<table>
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<th>Title</th>
<th>Authors</th>
<th>Company/Institution</th>
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<td>Analysis of a concrete face rockfill dam including concrete face</td>
<td>Gjorgi Kokalanov</td>
<td>Civil engineering school of</td>
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<td>loading and deformation using program package SOFiSTiK</td>
<td>Ljubomir Tančev</td>
<td>Skopje.</td>
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<td></td>
<td>Stevcho Mitovski</td>
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<td>Slobodan Lakočević</td>
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<td>DIA NA Analysis of a concrete faced rockfill dam</td>
<td>Gerd-Jan Schreppers</td>
<td>TNO DIA NA, Delft NL.</td>
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<td></td>
<td>Giovanna Lilliu</td>
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<td>A CFRD case using 3D modelling</td>
<td>C. Nieto</td>
<td>Tractebel Engineering-Coyne Et</td>
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<td></td>
<td>J-C. Philippe</td>
<td>Bellier. Gennevilliers Cedex,</td>
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<td>M. Werst</td>
<td>France.</td>
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<td>P. Anthiniac</td>
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<tr>
<td>Analysis of a concrete face rockfill dam including concrete face</td>
<td>C. Marulanda</td>
<td>INGETEC, Colombia.</td>
</tr>
<tr>
<td>loading and deformation</td>
<td>E. Leon</td>
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In general terms, it was concluded from the 10th Benchmark workshop that predicting rockfill behaviour relatively well is possible with the available numerical models. The main difficulty when modelling the CFRD behaviour is the interaction between the rockfill and the structural elements (i.e., face, plinth, joints, curb).
PORCE III: GEOLOGY AT DAM SITE. PLAN VIEW
PORCE III: GEOLOGY AT DAM SITE

Diagram showing various geological features and infrastructure at a dam site, including:
- Headrace Tunnel
- Excavation Surface
- Spillway
- Dam Crest
- Drainage Gallery
- Exploration Galleries
- Diversion Tunnel
- Subhorizontal Shear Zones

Key geological units:
- Quartzitic/graphitic schist
- Quartzitic/schist

Legend:
- Pink: Quartzitic/graphitic schist
- Light Pink: Quartzitic/schist
PORCE III: TYPES OF SCHISTS
PORCE III: TYPES OF SCHISTS
PORCE III: TYPES OF SCHISTS
PORT III: SOURCE – SPILLWAY ZONES

Table:

<table>
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<th>Zonas</th>
<th>Fases</th>
<th>A1</th>
<th>A2</th>
<th>B</th>
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<tr>
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<td>El. 695.0</td>
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<td>El. 675.0</td>
<td>El. 670.0</td>
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<td>4</td>
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<td>El. 645.0</td>
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<td>El. 640.0</td>
<td></td>
<td>El. 543.0</td>
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</table>
PORCE III: SPILLWAY EXCAVATION

Legend
- Qcol + Sr
- Pes-IIA
- Pes-IIIB
- Pes-III
- Pes-Graftoso

Excavation at EL. 680 msnm

Geo Virginia - 2015
PORCE III: PLATE LOAD TEST ON TRIAL FILL
Material 70% IIB+30% IIA.
Source: temporary deposits from spillway Zone A2
10.18.2007
PORCE III: PLACEMENT OF 3D ZONE MATERIAL

Material 70% I1B + 30% I1A. (Graphite schist).

10.18.2007
PORCE III: SLUICING OF 3D ZONE MATERIAL
Material 70% IIB + 30% IIA.

Source: directly from excavation in Zone A2 between levels 770 & 760.
PORCE III: ZONES 3D & 3C. DAM FILL AT EL. 555,7

Material 70% IIB + 30% IIA.
PORCE III: SLUICING OF 3D ROCKFILL
PORCE III: DOWSTREAM SLOPE. END OF STAGE 1
Truck identification based on material transported

Material degradation due to spreading process

Material 100% Type IIB

2008/03/06
PORCE III: STAGE 2 ZONES 3D & 3C

Zones 3D and 3C

CrossArm

Zones 2B & 4: filter material

RMV + PO

Material for zone 3C

RMV + PO

Material for Zone 3D

Material for Zone 3D

2008/03/06

Geo Virginia - 2015
PORCE III: DAM FILLS. CONSTRUCTION OF STAGE 2

STAGE 2

Mix Material
50% tipo III Underground. +
50% tipo III spillway.

Filter Zone 4

Filter Zone 2B

EL. 561.70

Zona 3D

Zona 3C

Zona 3B

EL. 561.10

2008/02/22
PORCE III
Interfaces

- Interface between slabs with compressible joints and tension joints
- Interface between concrete face and curb
- Interface between curb and rockfill
- Interface rockfill and foundation
- Interface between plinth and concrete face (perimeter joint)
- Project: Porce III (Col)
- Dam height: 150m
- Crest length: 330m
- $A/H^2$: 2.4

No compressible joints

$E = 60 \text{ MPa}$

$19.4 \text{ MPa}$

$5$ compressible joints

$10.8 \text{ MPa}$
COMPRESSIBLE VERTICAL JOINT

COMPRESSIBLE JOINT DETAIL IN INTERIOR SLABS

- Anchor bolts Ø0.95cm @ 40cm Length=7.5cm (stainless steel)
- PVC band Thickness=0.6cm Width=50cm
- Pre-molded liquid polyurethane or similar
- Liquid polyurethane or similar
- Class H concrete slab (reinforcement not shown)
- Adherent material (epoxy resin + sand)
- Design Line
- Concrete curb
- Copper seal Type A1 Width=55cm
- Mortar bed
- Thickness varies from 30.0cm to 60.0cm
- PVC band Thickness=0.6cm Width=50cm
- 3.0cm 14.0cm 14.0cm
- 3.0cm 4.5cm
- 13.25cm 13.25cm 50cm
- Geo Virginia - 2015
LABORATORY TEST ON COMPRESSIBLE MATERIAL
LABORATORY TEST ON COMPRESSIBLE MATERIAL
PROGRESS WORKS OF CONCRETE FACE
SLIDING SLAB PROCESS BETWEEN EL. 640 AND EL. 683
CONCRETE FACE – COMPRESSIBLE JOINTS
CONCRETE FACE – COMPRESSIBLE JOINTS
JOINT MOVEMENTS

Compressible Joints

(+): Opening of the Join
(-): Closing of the Join

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INSTRUMENTATION RESULTS

STRESS – X DIRECTION [MPa]

(-) Compression
(+) Tension

Geo Virginia - 2015
INSTRUMENTATION RESULTS

STRESS – Y DIRECTION [MPa]

(-) Compression
(+) Tension
NUMERICAL ANALYSIS

A. Rockfill settlement

U, [m]

Max = 1.02 m

B. Concrete face deformation

U, UB [m]

Max = 0.34 m

C. Concrete face stresses. Slope direction

6, S22 [MPa]
[SLOPE DIRECTION]

Max = 9.9 MPa

D. Concrete face stresses. Horizontal

S, S11 [kPa] [HORIZONTAL]

Max = 3.6 MPa

Compressible Joints
STRESS – X DIRECTION [MPa]
YEDIGOZE
MAZAR DAM - ECUADOR
MAZAR DAM
MAZAR COMPRESSIBLE JOINT
MAZAR COMPRESSIBLE JOINT
LA YESCA DAM
LA YESCA DAM

Geo Virginia - 2015
LA YESCA - DAM

Spillway
Screen
Substation
Concrete Face
Intake
LA YESCA - DAM
LA YESCA - DAM
LA YESCA - DAM
LA YESCA - DAM
LA YESCA DAM
La Yesca - Dam

General View of the Dam
May 2012
LA YESCA - DAM
Objective of the analysis:
- Estimate Stress – strain – deformations behavior of the concrete face
- Deformation of the fill
- Joint displacements of the joints
- Maximum deformations at the crest of the dam
- Dynamic behavior of the dam
1. Geometry of the model

1.1. Interfaces

Foundation level embedded fills and plinths
2. Constitutive Models

2.3. Interface behavior

<table>
<thead>
<tr>
<th>Interface</th>
<th>Interface Type</th>
<th>Normal Computation</th>
<th>Tangential Computation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Losa-Losa Juntas</td>
<td>Special Elements of contact</td>
<td>Rigid</td>
<td>Friction Model</td>
</tr>
<tr>
<td>Losa-Losa Juntas a tensión</td>
<td>Special Elements of contact</td>
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<td>Friction Model</td>
</tr>
<tr>
<td>Losa-Plinto</td>
<td>Special Elements of contact</td>
<td>Rigid</td>
<td>Friction Model</td>
</tr>
<tr>
<td>Losas-Bordillo</td>
<td>Special Elements of contact</td>
<td>Rigid</td>
<td>Friction Model</td>
</tr>
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<td>Bordillo-Enrocado</td>
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<td>Rigid</td>
<td>Friction Model</td>
</tr>
<tr>
<td>Enrocado-Muro</td>
<td>Special Elements of contact</td>
<td>Rigid</td>
<td>Friction Model</td>
</tr>
</tbody>
</table>

- Normal Compression - Rigid
- Normal Compression - Constant Rigidity
- Tangential Compression - Friction Model
3. Results

3.1. Analysis under static conditions

a. Deformation Modulus at the end of construction

b. Settlement of the fill

67cm

21cm

c. Displacement normal to the face

67cm

21cm
3. Results

3.1. Analysis under static conditions

c. Stresses in the horizontal direction (S11) [kPa]

d. Stresses along the slope [kPa]
3. Results

3.1. Analysis under static conditions

e. Minor principal stresses (compression) [kPa]

f. Mayor principal stresses [kPa]
LA YESCA - DAM
La Yesca Dam: Face Slab deformation (Sep 30/2013)
Figure 6. Settlement contours at Aguamilpa Dam as of October 2011 (first filling is included)
Figure 7. Settlement contours as of May 2011 measured at El Cajón Dam (first filling is included).
Figure 8. Settlement contours as of November 2012 measured at La Yesca Dam (first filling).
SOGAMOSO DAM

- 190 meters tall.
- 345 meters wide.
- 8,5 million cubic meters.
## DAM ZONES

<table>
<thead>
<tr>
<th>Zone</th>
<th>Volume [m$^3$]</th>
<th>Description</th>
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<tbody>
<tr>
<td>2A</td>
<td>31.000</td>
<td>Processed gravel</td>
</tr>
<tr>
<td>2B</td>
<td>289.600</td>
<td>Processed gravel</td>
</tr>
<tr>
<td>3A</td>
<td>4'103.600</td>
<td>Natural gravel</td>
</tr>
<tr>
<td>3B</td>
<td>2'307.300</td>
<td>Spillway rockfill</td>
</tr>
<tr>
<td>3C</td>
<td>1'293.500</td>
<td>Spillway rockfill</td>
</tr>
<tr>
<td>3D</td>
<td>128.000</td>
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</table>

### Plinth Detail

- **Concrete face**
- **Zona 1A**
- **Zona 2B**
- **Zona 3B**
- **Zona 3C**
- **Zona 3D**
- **RCC**
- **Losa perimetral**
- **Zona 1B**
- **Cara de contorno**
- **Zona 2A**
- **Zona 2B**
- **Zona 3A**
- **Bordillo**
- **Plinth detail**
CONSTRUCTION STAGES OF THE DAM

Duration: 22.5 months
Total Volume: 8’153.000 m³
Average efficiency: 362.360 m³/mes

<table>
<thead>
<tr>
<th>Stage</th>
<th>Volume [m³]</th>
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<tbody>
<tr>
<td>I - Down stream</td>
<td>568.300</td>
</tr>
<tr>
<td>I - Up stream</td>
<td>793.000</td>
</tr>
<tr>
<td>II</td>
<td>1’925.000</td>
</tr>
<tr>
<td>III</td>
<td>1’282.700</td>
</tr>
<tr>
<td>IV</td>
<td>992.000</td>
</tr>
<tr>
<td>V</td>
<td>1’047.000</td>
</tr>
<tr>
<td>VI</td>
<td>986.000</td>
</tr>
<tr>
<td>VII</td>
<td>511.000</td>
</tr>
<tr>
<td>VIII</td>
<td>48.000</td>
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FILL DISPLACEMENTS

$E_{3A} = 130$ MPa $E_{3B} = 50$ MPa $E_{3C} = 60$ MPa

$E_{3A} = 210$ MPa $E_{3B} = 50$ MPa $E_{3C} = 60$ MPa

$E_{3A} = 130$ MPa $E_{3B} = 100$ MPa $E_{3C} = 60$ MPa

$E_{3A} = 210$ MPa $E_{3B} = 100$ MPa $E_{3C} = 60$ MPa
FILL ZONES

Concrete face

Zona 3A  Zona 3B  Zona 3C  Zona 3D

RCC

CONCRETE FACE JOINT DISTRIBUTION

15 m

e = 0.3 + 0.003H

POSSIBLE COMPRESSIBLE JOINTS
MAXIMUM HORIZONTAL STRESS

\[ E_{3A} = 130 \text{ MPa} \quad E_{3B} = 50 \text{ MPa} \quad E_{3C} = 60 \text{ MPa} \]

\[ E_{3A} = 210 \text{ MPa} \quad E_{3B} = 50 \text{ MPa} \quad E_{3C} = 60 \text{ MPa} \]

\[ E_{3A} = 130 \text{ MPa} \quad E_{3B} = 100 \text{ MPa} \quad E_{3C} = 60 \text{ MPa} \]

\[ E_{3A} = 210 \text{ MPa} \quad E_{3B} = 100 \text{ MPa} \quad E_{3C} = 60 \text{ MPa} \]
MAXIMUM STRESS ALONG THE CONCRETE FACE (DOWNWARD DIRECTION)

$E_{3A} = 130$ MPa  $E_{3B} = 50$ MPa  $E_{3C} = 60$ MPa

$E_{3A} = 210$ MPa  $E_{3B} = 50$ MPa  $E_{3C} = 60$ MPa

$E_{3A} = 130$ MPa  $E_{3B} = 100$ MPa  $E_{3C} = 60$ MPa

$E_{3A} = 210$ MPa  $E_{3B} = 100$ MPa  $E_{3C} = 60$ MPa
SOGAMOSO DAM
SOGAMOSO DAM
RIGHT PLINTH EXCAVATIONS
CONCRETE

Final pouring – Section 16
Cu = 108
K = 9 \times 10^{-04} \text{ m/s}
modulus of compressibility BY IN SITU LOAD PLATE TESTS
ZONE 3 A (Eav = 147 Mpa)

Modulo de deformabilidad \( E_{sec} \) (MPa)
Zona 3A

- Valor promedio (146.8 Mpa)
- Placa de Diámetro = 46.5 cm
- Placa de Diámetro = 75.5 cm
SOGAMOSO DAM
SOGAMOSO DAM

JUNE 2013
CONSTRUCTION PROGRESS - CONCRETE FACE
AERIAL VIEW OF THE DAM
AERIAL VIEW OF THE DAM
AERIAL VIEW OF THE DAM
SETTLEMENTS

Fill settlements were measured by hydraulic cells and magnetic ring settlement gauges
CROSS SECTION - SETTLEMENT AT EL. 250,00 m.a.s.l.

End of construction
After filling (Dec 31 2014)
CROSS SECTION - SETTLEMENT AT EL. 290,00

End of construction
After filling (Dec 31 2014)
MAXIMUM SECTION - SETTLEMENT AT EL. 206,50 m.a.s.l

-66.2 cm
-101.9 cm
-84.2 cm
-98.6 cm
-126.2 cm
-153.2 cm
-197.0 cm
-211.1 cm
-315.2 cm
-329.0 cm

End of construction
After filling
(Dec 31 2014)
MAXIMUM SECTION - SETTLEMENT AT EL. 250

End of construction
After filling (Dec 31 2014)
SETTLEMENT OF NEAREST CELLS TO THE CONCRETE FACE AT END OF CONSTRUCTION AND AFTER FILLING

End of construction

After filling
MODULUS DURING CONSTRUCTION

\[ E_{rc} = \gamma H d_1 / \delta_s \]
MODULUS DURING RESERVOIR FILLING $E_{rf}$ (Fitzpatrick et al, 1985)

<table>
<thead>
<tr>
<th>CELL</th>
<th>Elevation (m.a.s.l)</th>
<th>Normal displacement $\delta_n$ (cm)</th>
<th>Modulus $E_{rf}$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>CA - 02</td>
<td>206,5</td>
<td>35,7</td>
<td>228</td>
</tr>
<tr>
<td>CA - 18</td>
<td>250</td>
<td>44,1</td>
<td>184</td>
</tr>
<tr>
<td>CA - 34</td>
<td>290</td>
<td>16,9</td>
<td>180</td>
</tr>
</tbody>
</table>

$$E_{rf} = \gamma_w \cdot h \cdot d_2 / \delta_n$$
Strain Gauge

Eje X: Along the concrete face
Eje Y: Horizontal to the concrete face,
Eje Z: Perpendicular to the concrete face
Geo Virginia - 2015

CONCRETE FACE
JOINT METER THREE DIMENSIONAL INSTRUMENT

Sensor in X axis: Measures the displacement of the joint between the slab and the plinth (shear)
Sensor in Y axis: Measures the opening or closing of the joint with the plinth
Sensor in Z axis: Measures if the slabs goes up or down with respect to the plinth
**Geo Virginia - 2015**

**CONCRETE FACE**
**JOINT METER  TWO DIMENSIONAL INSTRUMENT**

**Slab 25**

**2MJ3**

**Y**

**Z**

**Plinth**

**Medidor de junta 2MJ – en talón de la junta perimetral**

Sensor in **Y axis**: measures the opening or closing of the slab with respect to the plinth.

Sensor in **Z axis**: Measures if the slabs goes up or down with respect to the plinth.
Horizontal Stresses (parallel to the dam axis) obtained from strain gauges placed at EL. 226.04 masl

Geo Virginia - 2015
Horizontal Stresses (parallel to the dam axis) obtained from strain gauges placed at EL. 259,25 masl
Horizontal Stresses (parallel to the dam axis) obtained from strain gauges placed at a EL. 292.47 masl
Horizontal stress contours 31 de marzo de 2015 [MPa].
Stresses along the concrete face plane at EL. 226,04 msnm
Stresses along the concrete face plane at 259,25 msnm

![Graph showing stresses and reservoir levels over time](image)

Geo Virginia - 2015
Stresses along the concrete face plane at EL. 292.47 msnm
Perpendicular stress contours 31 de marzo de 2015 [MPa].
Joint total displacements at max. water elevation level 311 masl
Main findings are:

• Perimeter joints movements perpendicular to the face were larger than the openings. This confirms observed behavior in another dam with almost vertical abutments.[14]

• Maximum movement perpendicular to the face close to the center of the canyon was about 30 cm.

• Openings and closing along the same joint varied with elevation.
EL QUIMBO PROJECT - COLOMBIA

- Project: El Quimbo (Col)
- Height of the dam: 160m
- Length of crest: 659m
- A/H²: 4.74
EL QUIMBO HYDROELECTRIC PROJECT
Plinth Excavations

Plan View: Excavations for 11 sectors of the plinth
HYDROELECTRIC PROJECT EL QUIMBO

Plinth: Excavations for the left abutment

Plan view: Excavations of the left abutment for 6 sectors of the plinth
HYDROELECTRIC PROJECT EL QUIMBO

Plinth: Excavations for the right abutment

Plan view: Excavations of the right abutment for 4 sectors of the plinth
HYDROELECTRIC PROJECT EL QUIMBO - Instrumentation
HYDROELECTRIC PROJECT EL QUIMBO – Dam Sections
Instrumentation
CONCRETE FACE
VERTICAL JOINTS – DETAILS

- Exterior Vertical Joints
- Interior Vertical Joints
- Exterior Vertical Joints
- Especial Vertical Joints

Geo Virginia - 2015
HYDROELECTRIC PROJECT EL QUIMBO
HYDROELECTRIC PROJECT EL QUIMBO
HYDROELECTRIC PROJECT EL QUIMBO
HYDROELECTRIC PROJECT EL QUIMBO – Dam Sections Instrumentation
HYDROELECTRIC PROJECT EL QUIMBO Instrumentation

Geo Virginia - 2015
HYDROELECTRIC PROJECT EL QUIMBO
HYDROELECTRIC PROJECT EL QUIMBO
• The design and development of CFRD dams, have been based primarily on precedent and empiricism, however, recent incidents have shown that the extrapolation of precedent with the current procedures can have serious consequences. The framework described provides a rational and systematic approach for evaluating the rockfill properties that complemented with the use of numerical methods to predict the response of CFRD’s which will consequently contribute to the proper evolution of these dams. It will enhance and integrate precedent with numerical modeling.
Based on the observed behavior, it can be concluded that the main problem in the dams that have behaved adversely was the characteristics of the rockfill, which confirmed that a “good” rockfill is not defined by the existence of hard fill particles but by its gradation, key in obtaining a less deformable rockfill. This issue was well recognized several decades ago by Marsal, however, is likely that given the pressure to further reduce costs and time for this already economical dam, this fundamental knowledge on rockfill behavior was somehow overlooked. The adequate processing of a rockfill, including gradation, sluicing and compaction are essential to obtain an adequate behavior of a rockfill dam.
CONCLUSIONS

- The physical mechanisms involved in the interaction of the different structural components of a CFRD dam are a very complex to model; however, with the current computational capabilities available, the development of very sophisticated analyses can aid considerably the design process of CFRD dams. The analyses are quite useful to evaluate the effectiveness of different mitigation measures to alleviate stresses in the concrete face. Nevertheless, the results of current numerical analysis cannot be taken as absolute and precise values. The analyses should point up tendencies and estimates of stress – strain behavior of the different components of the dam, so that engineers with good judgment can make further decisions.
ILISU PROJECT – TURKEY