Reconstrucción de la presa de abastecimiento Camará
Rebuilding Camará water supply dam

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Resumen
La presa Camará construida en Brasil es una estructura de concreto compactado con rodillo con 50 m de altura, con fundaciones en granitos y migmatitas que presentan juntas de alivio que se desarrollan paralelas a la superficie topográfica. La estructura en la ladera izquierda se rompió en el año 2004, debido a la presencia de esas discontinuidades.

La reconstrucción empezó en 2011 y el nuevo modelo geomecánico desarrollado se presenta con grupos de juntas fuertemente meteorizadas, que todavía permanecen con estructuras de la roca original, formando una roca muy blanda.

Hasta una determinada profundidad la presencia de las discontinuidades compromete la estabilidad al deslizamiento, pues el ángulo de roce disponible fue interpretado como 35º sin cohesión. Estos parámetros son insuficientes para garantizar las condiciones establecidas en los criterios de Proyecto. Como consecuencia fue necesaria la construcción de llaves de concreto en la parte aguas abajo de las estructuras.

Abstract
Camará dam is a Brazilian 50 m high roller compacted concrete structure founded on granite and migmatites with expressive weathered exfoliation joints, which develop parallel to the topography.

In 2004 the left abutment failed due to the presence of such exfoliation joints, which caused the dam to collapse, emptying the reservoir.

The rebuilding process started in 2011 and the new geomechanical model came up with a set of joints, parallel to the surface and some faults with a high degree of weathering producing a soft material that retains rock mass relics.

To a certain depth the presence of those features compromise the sliding stability of the dam, since the available friction angle for the features was taken as 35º, with no cohesion. These parameters are insufficient to fulfil the design criteria requirements. As a consequence, concrete surface shear keys were needed downstream of all concrete blocks.
1 INTRODUCTION

The Camará dam is owned by the government of the state of Paraíba, Northeast region of Brazil (Figure 1) on the Riachão River. It is a 50 m high rolled compacted concrete structure designed to supply 2,4 m³/s. The spillway will discharge 529,11 m³/s. The 14 km long reservoir will store 26,5x10⁶ m³.

It failed in June, 2004, due to erosion of heavily weathered exfoliation joints occurring within the sound rock mass, a few meters below the foundation. According to Soft Rocks Commission approach, those joints can be considered soft rocks in terms of foundation for concrete dam structures.

The drain holes of the internal drainage system, were drilled through the sandy clayey material, which, due to the high gradient, started a piping process, washing out all the joint gouge through the drainage galleries. Huge hollow zones appeared below the foundation, removing all the support for the concrete structure, which eventually came to failure. (Figures 2 and 3).

Little after the situation shown in Figure 3 the concrete structure failed completely. Local investigators reported casualties, property loss and overall damages.

After cleaning, the discontinuities outcropped as shown in Figure 4.

2 INVESTIGATION AND GEOMECHANICAL MODEL

The rebuilding process started in 2011 with a new program of investigation and analyses. The investigation program encompassed surface mapping, coring, borehole videotaping, water loss tests, and lab tests of the joints gouge materials.

The new phase of investigation demanded 43 boreholes, with coring and water loss tests in all of
them. Of those, eleven holes were recorded with digital camera.

In tropical climate the intermediate product of this weathering is a mass of disaggregated particles, basically quartz, altered mica, feldspars and accessories as shown in Figure 5. In some cases, clay is also present.

Figure 5. Macro mode photograph of weathered migmatite.

Videotaped holes showed one of the discontinuities as illustrated in Figure 6.

Figure 6. Digital image of the borehole walls, sowing the discontinuity which correspond to one of the exfoliation joints.

After cleaning the foundation surface of the left bank, some exfoliation joints had their gouge materials sampled, and taken to the laboratory to perform characterization and direct shear tests, mainly to gather some indication of shear strength and stiffness.

Disturbed and undisturbed samples were submitted to direct shear tests and the results showed variations from 39º of friction angle for the sandy samples to 22º of residual strength to clayey samples. All tests did not record any cohesion.

The stiffnesses of the joints were recorded whenever possible, resulting in the values presented in Table 1.

Table 1. Joint stiffnesses measured in laboratory

<table>
<thead>
<tr>
<th>Test</th>
<th>ks</th>
<th>kt</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3,00</td>
<td>0,04</td>
</tr>
<tr>
<td>2</td>
<td>2,73</td>
<td>0,06</td>
</tr>
<tr>
<td>3</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>4</td>
<td>N</td>
<td>0,09</td>
</tr>
<tr>
<td>5</td>
<td>N</td>
<td>0,08</td>
</tr>
<tr>
<td>6</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>Mean</td>
<td>2,87</td>
<td>0,07</td>
</tr>
</tbody>
</table>

Unit GPa/m

The final geomechanical model came up with a set of joints, parallel to the surface and some faults with a high degree of weathering.

Considering the average distribution, persistence, filling and roughness of the joints, their strength was globally taken as 35º of friction, with no cohesion. (Figure 7).

Figure 7. Vertical cross section through dam axis showing the model adopted to perform 2D stability analysis.

The geomechanical model based on this section considered the following parameters:

- Foundation migmatite sound rock mass: deformability modulus = 20 GPa; Poisson = 0,25.
- Disturbed surface rock mass: deformability modulus = 15 GPa; Poisson = 0,25.
- Exfoliation joints: average friction angle 35º, cohesion = 0,0 kN/m²; shear stiffness ks = 0,07 GPa/m; normal stiffness kn = 2,87 GPa/m.
- Contact concrete x sound rock: friction angle 45º; cohesion = 200 kN/m².

Hundreds of water loss tests registered a rock mass almost impervious with its permeability varying from $5 \times 10^{-5}$ cm/s to practically impervious. Exceptionally, in a few spots where the weathered joints were sampled, some total loss was recorded.

3 MODELLING

Down to a certain depth the presence of those features compromise the sliding stability of the
dam. The joints parameters are insufficient to fulfill the design criteria requirements.

By means of numerical models (SAP 2000 V.15), the sliding stability was checked repeatedly, for various depths, including the downstream thrust of the rock, that is one of the remaining rock mass sliding on a surface with a certain stiffness (0.07 GPa/m for shear) taken from references and lab tests.

As a result, to a certain depth, concrete surface shear keys were needed downstream of the existing and new blocks, which was a function of the concrete block height.

In order to calculate the safety against the sliding through the discontinuities in the foundation rock mass, a two-dimensional mathematical model (FEM) was assembled to determine the stress distribution and magnitude to develop the limit equilibrium analysis as stated in the design criteria. Figure 8 shows the two-dimensional mathematical model and its main features. The model ran with the traditional software SAP2000, with linear elastic behavior.

![Figure 8 – Mathematical model (2D)](image)

In order to better view the resulting stresses distribution along the joint within the foundation, the tangential ($\tau$) and normal stresses ($\sigma$) were plotted (Figure 9) and it is clearly observed that the stiffer surface (contact between the concrete and rock in the shear key) takes much more tangential stress, leaving the joint with less loads.

![Figure 9 – Tangential stress, normal stress and $\phi_{mob}$ at potential sliding plane.](image)

The idea of mobilized friction angle $\phi_{mob}$ indicates the variation of the friction angle that has to be available in order to accomplish the equilibrium. The $\phi_{mob}$ can be derived as:

$$\phi_{mob} = \arctan ((\tau - c) / (\sigma - u))$$  \hspace{1cm} (1)

Where $c$ is cohesion and $u$ the uplift.

In border elements some tangential stresses are higher than the available strength and had their elastic properties reduced by means of plastification.

The compressive stresses applied on the vertical plane CD performed the passive rock pressure in the downstream toe of the dam as shown in Figure 10.

![Figure 10 – Detail of the concrete shear key. Horizontal compressive stresses at the vertical downstream face.](image)

The sliding stability evaluation was developed to other blocks besides the spillway by
determining the shear friction factor, as defined below:

\[ SFF = \frac{(Rf + Rc)}{F} \geq 1.0 \]  

(2)

With:

\[ Rf = \frac{(N_{AB} - U_{AB}) \tan \phi_{AB} + (N_{BC} - U_{BC}) \tan \phi_{CD}}{\gamma \phi} \]  

(3)

\[ Rc = \frac{(c_{AB} A_{AB} + c_{BC} A_{BC})}{\gamma c} \]  

(4)

\[ F = F_U - F_D - Ep \]  

(5)

The equilibrium forces are shown in Figure 11.

Figure 11 – Equilibrium forces.

where:

\[ W_C + W_R + W_S = N_{AB} + N_{BC} \]  

(6)

\[ N_{AB} \text{ and } N_{BC} = \text{Normal forces in AB and BC segments.} \]

\[ U_{AB} \text{ and } U_{BC} = \text{Uplifts in AB and BC segments.} \]

\[ A_{AB} \text{ and } A_{BC} = \text{Areas in AB and BC segments.} \]

\[ \phi_{AB} \text{ and } \phi_{CD} = \text{Friction angles in AB and BC segments.} \]

\[ c_{AB} \text{ and } c_{CD} = \text{Cohesions in AB and BC segments.} \]

\[ F_U \text{ and } F_D = \text{Hydraulic thrusts.} \]

\[ Ep = \text{Passive rock pressure.} \]

\[ \gamma \phi = \text{Safety factor for friction.} \]

\[ \gamma c = \text{Safety factor for cohesion.} \]

For the complete verification of the sliding stability in each cross section dam, several analyses have been done considering the Normal Load Condition (NLC), the Exceptional Load Condition (ELC) and the Limit Load Condition (LLC). For each of these three load cases was adopted for the passive rock pressure (Ep) the following percentages of participation in the model as shown in Table 2.

![Diagram](image)

Table 2 – Percentages of Ep and safety factors

<table>
<thead>
<tr>
<th>Case</th>
<th>EP</th>
<th>( \gamma \phi )</th>
<th>( \gamma c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>NLC</td>
<td>20</td>
<td>1.5</td>
<td>3.0</td>
</tr>
<tr>
<td>ELC</td>
<td>40</td>
<td>1.1</td>
<td>1.5</td>
</tr>
<tr>
<td>LLC</td>
<td>80</td>
<td>1.1</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Six loading cases were considered according to the method of limit equilibrium, combining the reservoir water level, the incidence of seismic efforts and drainage system efficiency. The shear friction factor results are presented in Table 3. Various loading cases without shear key resulted in values smaller than the necessary according to design criteria. With the key, the controlling case is NLC which delivered a safety factor slightly above 1 (Eletrobrás recommendation), as needed.

Table 3 – Shear friction factor results

<table>
<thead>
<tr>
<th>Case</th>
<th>UWL</th>
<th>DWL</th>
<th>Seismic (1)</th>
<th>Operating Drain (2)</th>
<th>SFF (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(1)</td>
<td></td>
<td>no key</td>
</tr>
<tr>
<td>NLC</td>
<td>461.00</td>
<td>417</td>
<td>NO</td>
<td>YES</td>
<td>0.86</td>
</tr>
<tr>
<td>ELC1</td>
<td>461.00</td>
<td>417</td>
<td>YES</td>
<td>YES</td>
<td>1.03</td>
</tr>
<tr>
<td>ELC2</td>
<td>424.26</td>
<td>418</td>
<td>NO</td>
<td>YES</td>
<td>1.10</td>
</tr>
<tr>
<td>ELC3</td>
<td>461.00</td>
<td>417</td>
<td>NO</td>
<td>NO</td>
<td>0.87</td>
</tr>
<tr>
<td>LLC1</td>
<td>424.26</td>
<td>418</td>
<td>NO</td>
<td>NO</td>
<td>0.88</td>
</tr>
<tr>
<td>LLC2</td>
<td>461.00</td>
<td>417</td>
<td>YES</td>
<td>NO</td>
<td>0.80</td>
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<tr>
<td>LLC3</td>
<td>424.26</td>
<td>418</td>
<td>YES</td>
<td>YES</td>
<td>1.04</td>
</tr>
</tbody>
</table>

(1) \( \alpha_h=5\%g, \alpha_v=3\%g. \)

(2) According the Bureau of Reclamation Criteria.

(3) SFF_{min}=1.0 (Eletrobrás).

The analyses for all representative blocks showed that the shear key would be an adequate solution for the entire dam to achieve the design criteria, since ¼ of the dam were already built.

This text describes the spillway analysis; however, various calculation rounds were performed to optimize the length and depth of each shear key for abutment blocks, as well.

### 4 DESIGN AND CONSTRUCTION

In order to improve the seepage control, as additional safety, the drainage system was refurbished and an additional grout curtain was built. The grout holes were drilled from a slab built in the foundation, upstream of the dam face as shown in Figure 12. This slab develops all along the dam and is 5 m wide, minimum, 1 m
The existing drains, spaced every 3 m were washed and enlarged from 3” to 4”. The new ones were drilled in same way: spaced 3 m and 4” of diameter. The depth of the drains is approximately 40% of the dam height. A filtering drain is installed in each drain hole to avoid the fine particles of the weathered joints to be washed out.

The grout curtain was built by means of two lines 3 m apart of holes. The primary and secondary holes, spaced 12 m, were mandatory and considered exploratory. Depending upon the grout take on previous holes, tertiary holes were drilled in between the previous ones. The limit to introduce tertiary holes was 25 kg of cement/m.

For the primary holes the average take was 21 kg/m with standard deviation of 76 kg/m, due to higher takes at the exfoliation joints (maximum allowed takes for primary holes was 25 kg/m). From almost 400 stretches to that date, 7% needed tertiary holes, and very few, quaternary holes.

For the secondary holes the average take was 21 kg/m with standard deviation of 30 kg/m. From almost 400 stretches to that date, 3% needed tertiary holes, and very few, quaternary holes.

The dam construction started with the debris cleaning on the whole area. The surface mapping started at this point.

After the design setting, the area of the shear key downstream of the spillway started to be excavated by means of cable cut to avoid major damage in the remaining concrete.

As soon as the excavation reached the bottom, the plain concrete started to be poured. In the remaining blocks the older concrete had to be partially cut to give room to the shear key, while in the left bank, in the failure area, the shear keys were incorporated to the new structure. Figures 13 to 16 illustrate this sequence.
the concrete and the foundation. Part of the excavation of the left bank appears in the right side of the picture. Some exfoliation joints can be seen in the rock walls.

![Figure 15](image1.png)

**Figure 15.** Upstream wall of the excavated shear key in the left bank. A set of exfoliation joints was intercepted by the excavation. The shear key is partially concreted.

![Figure 16](image2.png)

**Figure 16.** After cleaning the upstream area of the dam, the slab was built. In the picture one can see the watertight device being installed. The contraction joint in the slab itself was made watertight, as well. The minimum thickness of the structure was 1 m. Its width varied from 5 to 10 m, depending on the condition of joints outcropping.

By the date of preparation of this paper, a full set of instruments, basically 3 multi rod extensometers and 14 piezometers, were already purchased.

At this point, the whole concrete procedure was considered finished (Figure 17).

The reservoir filling and commissioning are expected to take place at the end of this year of 2016.

![Figure 17](image3.png)

**Figure 17.** Aerial view of the refurbished dam in July, 2016.

## 5 CONCLUSION

The presence of exfoliation joints below the foundation of Camará concrete dam was an important issue in the failure event of the left bank.

The exfoliation joints still in foundation were considered Soft Rocks, and were of fundamental importance for modelling the dam and foundation assembly.

The use of shear keys was then considered, and the extensive surface mapping during excavation process confirmed the geomechanical model used for calculating the dam stability.

Finally, the calculated safety factors complied with the requirements of the approved design criteria.

## 6 REFERENCES


## 7 ACKNOWLEDGEMENTS

We acknowledge the construction consortium CRE & AGE for the assistance and permission to publish this paper.