# STRUCTURAL DESIGN OF A HYDROELECTRIC POWER PLANT INSERTED IN AN EMBANKMENT DAM

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**May 2017**

## Abstract

The main objective of this dissertation is to ensure the global and internal stability of the hydroelectric power plant superstructure, which includes the structural design and detailing of the reinforced concrete sections of the structural left wall, rooftop beams and interior slab.

This study focuses on a large dam with 45 [m] height and a transverse development of 22 [m]. Its reinforced concrete structure is classified as category 5, which corresponds to a structural class S6.

At a first stage, structural materials, actions, and load combinations are defined, elements are evaluated using simplified models, which allow to validate the initially geometries and to obtain a first approach of the structure behaviour and the reinforcement required.

The second stage introduces the three-dimensional models, which use a finite element method program. These models allow to get accurate results, that will be compared with previous ones.

In the final stage, it is presented the general arrangement drawings and the reinforcing detailing of the analysed structural elements, and the conclusions about the models and obtained results.

Throughout this thesis, it is followed the Portuguese regulations, except in the cases it is non-existent.

### Key-words

Gravity Dam | Reinforced concrete | Structural design | Stability | Cracking | Three-dimensional modelling

## 1. Introduction

Dams are man-made structures which interrupt the natural flow of rivers. These structures are particularly important for the modern world, since they allow the public supply of large metropolises, as well as the production of electricity and agricultural or industrial production in the surrounding areas. it is also a proven mechanism to prevent floods and it allows to manage river flows.

### 1.1. Main goals

The main goal of this dissertation is to perform a stability analysis of the structure and verify the global stability of the power plant structure and internal stability of some elements: the structural left wall, rooftop beams and interior slab. In this study, it is used the preliminary geometric definitions proposed by the supervisor, which are presented in the general arrangement drawings (1 to 4).

Firstly, it is made a three-dimensional model in AutoCAD 2016® [B] that guarantees the compatibility between different architect plants and allows to calculate the volume and geometric centre, which are important parameters for the following analysis.

Secondly, it is defined the loads and all the scenarios so that we could calculate the factors of safety for sliding, uplift and overturning, and the stresses in the foundation. Then, if these values meet the criteria, the geometry of elements is accepted.

The next stage is related with the verification of the internal stability (ULS and SLS) of the selected elements according to the results based on simplified model and three-dimensional finite element method.

Lastly, it is made a comparison between the result obtain from the different methods, and it also presented the reinforcement detail drawing in volume II (5-7) according to SAP2000® results.

### 1.2. Background

Water is an important natural resource that Man had always try to manage since the first civilizations.

The construction of dams allows the development of large metropolis and the management of river flows according to man's needs. The research about this topic resulted in the introduction of turbine in large dams for production of electricity, increasing the general interest in this kind of structures [1].

These structures may have different proposes, such as hydroelectric generation, stabilization water flow, irrigation, flood prevention, water supply and others.

The construction is a multidisciplinary project which must take in to account not only the type of structure, but also the relation between this and the environment around.

## 2. 3D Modelling

The Figure 1 illustrates the three-dimensional model generated in AutoCAD 2016® [B], which helps to better understand the geometric definition of the structure.

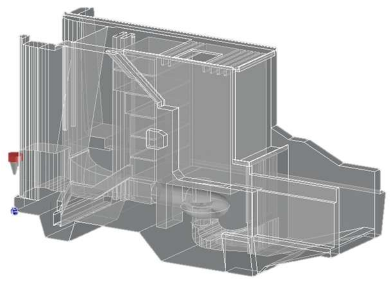

*Figure 1 – Three-dimensional model*

Furthermore, this model allows to obtain the volume of the structure and the centre of gravity.

## 3. Structural materials

In accordance with NP EN 1992-1-1 [2] and Portuguese Regulation [3], the choice of material is related with durability requirements, in order to verify the serviceability, strength and stability throughout its design working life, without significant loss of utility or excessive unforeseen maintenance.

According to NP EN 1992-1-1 [2], this structure is established as an important structure (category 5), which corresponds to a structural class S6. Due to this fact, the dam should be designed for a working life of 100 years.

In this project, it was selected a structural steel S500 because of its better performance in elements where the amount of reinforcement is controlled by the resistance to ULS. Concrete selection followed the norm NP EN 206-1 [4], which defined the minimum resistance class for concrete according its exposure class to corrosion induced by carbonation. The Table 1 resumes the selection for the different elements.

*Table 1 – Concrete specification*

| Elements | Specification |
|---|---|
| Interior Slabs | *C25/30 XC1(Pt) Cl 0,4 $D_{max}$22 S3* |
| Others | *C30/37 XC4(Pt) Cl 0,4 $D_{max}$22 S3* |

## 4. Design situations and combinations of actions

Portuguese Regulation established the scenarios that must be taken into account:

current scenario (CS) and failure scenario (FS) [3], [5].

Besides these, it may also be important to consider a limit scenario on this study.

### 4.1. Design situations

The current scenario (CS) corresponds to all the load combination with high probability of acting during the working life of structures [6]. The scenarios that match these criteria are:

I) $ATL + AHL$ – Normal hydrological conditions – Average level headwater (112,00 [m]) and average tailwater level (83,70 [m]);
II) $ATL + AHL + OBE$ – Normal hydrological conditions (previous scenario) and an operating basis earthquake.

The failure scenario (FS) is related with the combination of loads that presents a low probability of occurrence, such as:

III) $TFE + HFE$ – Probable maximum flood (PMF) – headwater at a flood elevation (113,00 [m]), and tailwater at a flood elevation (92,00 [m]);
IV) $TFE + HFE + MCE$ – Probable maximum flood combined with a maximum credible earthquake;
V) $ATL + AHL + damage\ drain\ curtain$ – Normal hydrological conditions (previous scenario) and an inoperative drain curtain.

The limit scenario (LS) corresponds to a combination of action with low probability of occurrence at the same time. One for this combination is:

VI) $TFE + HFE + damage\ drain\ curtain$ – Probably maximum flood and an inoperative drain curtain.

### 4.2. Limit states

According to NP EN 1990 [7], the design of the structure should distinguish and evaluate ultimate limit states (ULS) and serviceability limit state (SLS).

#### *4.2.1. Ultimate Limit States (ULS)*

In this project, the limit states included in the assessment of global stability are:

- Loss of equilibrium of the structure due to uplift (UPL);
- Loss of equilibrium of the structure due to overturning or sliding (EQU);
- Failure of the ground (GEO).

In this evaluation, Portuguese Regulation [3] and the others international documents used on this project [6], [8] impose we must calculate and guarantee factors of safety higher than the values presented in Table 2.

*Table 2 – Factors of safety for global stability*

| Safety Factors for Global Stability | | | |
|---|---|---|---|
| **Scenario** | **Sliding** $FSS_{\phi}$ | **Overturning** $FSO$ | **Uplift** $FSU$ |
| **CS** | 1,5 | 1,5 | 1,3 |
| **FS** | 1,2 | 1,2 | 1,1 |
| **LS** | 1,1 | 1,1 | 1,1 |

Moreover, NP EN 1990 [7] refers the need to ensure internal stability meets the requirements for internal failure or excessive deformation of the structure or structural members (STR), which are stablished by the following equations.

$$E_d \leq R_d \tag{1}$$

$$E_d = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{2}$$

#### *4.2.2. Serviceability Limit States (ULS)*

These limit states concern with the functioning of the structure, the comfort of the people and the appearance of the construction works [7]. The complete analysis involves the validation of equation 3 for stress limitation ($0{,}80 f_{yk}$ for steel and $0{,}60 f_{ck}$ for concrete), crack width control (0,30 [mm]) and maximum deformation, according to NP EN 1992-1-1 requirements (Table 3) [2].

$$E_d \leq C_d \tag{3}$$

$$E_d = \sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{4}$$

*Table 3 – Maximum deformation of concrete*

| Element | $\delta_{adm}\ [mm]$ |
|---|---|
| Interior slab | 24,8 |
| Interior beams V1 (V3) | 21 (12,6) |
| Roof Beam B (A) | 30 (16) |
| Roff Slab | 2,6 |
| Structural wall | 42,1 |

## 5. Actions

In this project were considered the following set of actions: dead load (self-weight in concrete); other permanent loads; live loads; hydrostatic pressure; uplift pressure and seismic action.

## 6. Global stability

The stability criteria for gravity dams is guaranteed when the following structure moves are verified. The static equilibrium of the structure is usually related with the self-weight (value and position of gravity centre) [9].

### 6.1. Sliding

The sliding stability is based on the calculation of factor of safety, and in the comparison with its minimum value (Table 2).

This analysis follows the equation 5, in which it is considered an angle of internal friction ($\phi$) equals to 40º (basaltic foundation).

$$\frac{(\sum V - U)\times \tan\phi}{\sum H} \geq FSS_{\phi} \tag{5}$$

### 6.2. Uplift

The safety of the dam against uplift is based on equation 6. In this evaluation, it is only considered the vertical actions that are dead loads or unfavourable lived loads.

$$\frac{\sum V}{U} \geq FSU \tag{6}$$

### 6.3. Overturning

The overturning stability is based in the balance between the stabilizing and the destabilizing actions, whereas the seismic action has direction of water flow. The verification for this body movement is check by equation 7.

$$\left|\frac{\sum M_{stab}}{\sum M_{distab}}\right| \geq FSO \tag{7}$$

### 6.4. Analysis results – global stability

Table 5 resumes the verification against structure failure according to the previous criteria.

*Table 4 – Global stability results*

| Global stability | $FSS_{\phi}$ | | $FSU$ | | $FSO$ | |
|---|---|---|---|---|---|---|
| | Calc | Mín | Calc | Mín | Calc | Mín |
| S1 | 5,54 | 1,50 | 4,11 | 1,50 | 2,80 | 1,30 |
| S2 | 2,82 | | 4,04 | | 2,25 | |
| S3 | 5,54 | 1,20 | 3,19 | 1,20 | 2,57 | 1,10 |
| S4 | 2,25 | | 4,00 | | 2,05 | |
| S5 | 4,76 | | 2,86 | | 2,02 | |
| S6 | 4,81 | 1,10 | 2,48 | 1,10 | 1,89 | 1,10 |

### 6.5. Stresses in the foundation

The verification against stresses foundation failure is provided by equation 8, in which the ratio between maximum stresses and admission stresses should be higher than the value presented on Table 2.

$$\sigma_{max} \leq FS_{\sigma} . \sigma_{adm} \tag{8}$$

In this analysis is important to considered both seismic directions upwards (scenario $2^{(1)}$ and $4^{(1)}$) and downwards (scenario $2^{(2)}$ and $4^{(2)}$).

### 6.6. Analysis results – stresses

Table 5Table 5 resumes the verification and it is shown that the foundation is always under vertical compressive stresses.

*Table 5 – Stresses foundation results*

| Global stability | $FS_{\sigma}$ | | | | | |
|---|---|---|---|---|---|---|
| | $e/L$ | $\sigma_{mont}$ $[MPa]$ | $\sigma_{jus}$ $[MPa]$ | $\sigma_{adm}$ $[MPa]$ | Calc | Min |
| S1 | -0,04 | 0,45 | 0,28 | 14 | 31 | 5,0 |
| $S2^{(1)}$ | 0,00 | 0,36 | 0,34 | | 38 | |
| $S2^{(2)}$ | -0,01 | 0,40 | 0,35 | | 35 | |
| S3 | -0,05 | 0,43 | 0,23 | 14 | 32 | 1,5 |
| $S4^{(1)}$ | 0,01 | 0,32 | 0,38 | | 43 | |
| $S4^{(2)}$ | 0,00 | 0,37 | 0,38 | | 38 | |
| S5 | -0,01 | 0,34 | 0,29 | | 42 | |
| S6 | -0,02 | 0,32 | 0,26 | 14 | 44 | 1,5 |

## 7. Internal Stability

Once it is guaranteed the global stability of the structure, it is now necessary to evaluate the internal stability of the elements according to the requirements of NP EN 1992-1-1 [2] (ULS and SLS).

This analysis also allows to confirm rightness of the minimum resistance concrete class and cross section initial dimensions for each element.

## 7.1. Simplified models

Firstly, it is presented an internal stability analysis throughout simplified models which try to represent an approach behaviour and load distributions of the elements for ULS and SLS in the programmes Ftool® [C] and Gala® [D].

### 7.1.1. Roof beams

These set of elements has different geometric dimensions. The mains beams (defined by letter B in the tables) have a cross section with 0,50x1,50 [m$^2$] and a length of the span equals to 16 [m]. The secondary beams (defined by letter A in the tables) have a cross sections equals to 0,50x0,80 [m$^2$] and a 8,30 [m] span.

According to NP EN 206-1 [10], the exposure class takes a level XC3 and should be used a concrete cover equals to 45 [mm].

#### 7.1.1.1. Ultimate Limit State

In the ULS verifications it is considered a selft-weight of the beams, the selft-weight and live load applied on the top slab and the partial safety factors.

In Table 6 and Table 7 are exhibited the steel reinforcement rebar used on the design of these beams (longitudinal and transversal).

*Table 6 – Longitudinal reinforcement in the beam*

| Beams | $M_{sd}$ $[kNm]$ | $\mu$ $[-]$ | $A_s$ $[-]$ | $[cm^2]$ |
|---|---|---|---|---|
| B1 | 1 562,22 | 0,079 | $2\phi25 + 6\phi20$ | 28,67 |
| | -3 526,49 | 0,179 | $8\phi32$ | 64,34 |
| B2/B3 | 831,57 | 0,042 | $2\phi25 + 2\phi20$ | 15,71 |
| | -1 663,15 | 0,085 | $2\phi25 + 6\phi20$ | 29,45 |
| A2/A3 | 529,91 | 0,108 | $6\phi20$ | 18,85 |

*Table 7 – Transversal reinforcement in the beam*

| Beams | $\lvert V_{sd}\rvert$ $[kN]$ | $A_s/s$ Stirrups | $[cm^2/m]$ | $\sigma_c$ $[kPa]$ |
|---|---|---|---|---|
| B1 | 984,83 | ϕ10//0,10 (2R) | 15,76 | 3,60 |
| B2/B3 | 454,52 | ϕ10//0,30 (2R) | 5,24 | 1,66 |
| A2/A3 | 187,60 | ϕ10//0,30 (2R) | 5,24 | 1,37 |

It is also important to follow the minimum and maximum area of reinforcement established in NP EN 1992-1-1 [2] for both analysis.

#### 7.1.1.2. Serviceability Limit State

Table 8 resumes the SLS analysis for the roof beams. It is necessary to check if the bending moment in any section is higher than the cracking moment and if it happens, confirm if the crack width is less than 0,30 [mm].

This study also provides information about the deformation. The beams A and B had deflections equal to 10,6 and 10,4 [mm], which is less than the limit.

*Table 8 – SLS analysis for the roof beams*

| Beam | $M_{ELS}$ $[kN.m]$ | $\sigma_c$ $[MPa]$ | $\sigma_s$ $[MPa]$ | $w_k$ $[mm]$ |
|---|---|---|---|---|
| B1 | 1 099 | -9,35 | 229,94 | 0,23 |
| | -2 479 | -16,15 | 281,98 | 0,21 |
| B2/B3 | 584 | -6,15 | 153,04 | 0,05 |
| | -1 168 | -10,79 | 246,35 | 0,22 |
| A2/A3 | 372 | -11,47 | 271,73 | 0,27 |

### 7.1.2. Technical Slab

The structural analysis follows the principles from Bares tables [11] in which the slab is divided in different panels so that it is possible to match the models in the Bares tables. Then it is studied the bending moment and shear forces for each part and the envelope of stresses when the models are assembled together, which allow to check the overall reinforcement requirements.

In this chapter, it is only presented the structural analysis of the slab needs. The slab has a thickness equals to 0,35 [m] and an exposure class level XC1, so it should be used a concrete cover equals to 35 [mm].

#### 7.1.2.1. Ultimate Limit State

For this element, it is necessary to specify a minimum longitudinal reinforcement. In this case, it was defined as minimum reinforcement grid, 12 [mm] diameter and 10 [cm] of spacing. This solution it is a first approach and allow to minimize the reinforcement and to respect the minimum percentage defined in NP EN 1992-1-1 [2].

*Table 9 – Longitudinal reinforcement in the slab*

| Dir | Position | $M_{sd}$ $[kNm/m]$ | $\mu$ $[-]$ | $A_s$ $[-]$ | $[cm^2/m]$ |
|---|---|---|---|---|---|
| X | Span | 19,16 | 0,014 | ϕ12//0,10 | 11,31 |
| | Support | -44,05 | 0,033 | ϕ12//0,10 | 11,31 |
| Y | Span | 21,19 | 0,016 | ϕ12//0,10 | 11,31 |
| | Support | -53,06 | 0,039 | ϕ12//0,10 | 11,31 |

It was also performed a transversal reinforcement analysis according to the article 9.3.2. of the norm NP EN 1992-1-1 [2]. It was possible to noticed a resistance value equals to 146,97 [kN] which higher than the design value.

#### 7.1.2.2. Serviceability Limit State

The Table 10 summarizes the SLS analysis for the critical points in the slab, both directions and positions (supports and spans).

*Table 10 – SLS analysis for the interior slab*

| Dir | Position | $M_{ELS}$ $[kN.m/m]$ | $\sigma_c$ $[MPa]$ | $\sigma_s$ $[MPa]$ |
|---|---|---|---|---|
| X | Span | -37,78 | -4,52 | 122,28 |
| | Support | 15,09 | -1,86 | 48,76 |
| Y | Span | -36,48 | -4,76 | 122,28 |
| | Support | 13.40 | -1,80 | 44,86 |

This study allows to conclude that the bending moments are lower than the cracking moment and it is not expecting to have crack at all.

The deformation analysis for the slab it was performed using the slenderness ratio. In these elements, the values should be less than 20, which is verified (maximum value – 18,2).

### 7.1.3. Structural wall

This final element was analysed using a cantilever model subject to the soil pressure in the exterior surface. The wall has a XC4 exposure class which imposes a concrete cover with 50 [mm]. In this case, the model simplifies the boundary conditions but it remains a good first approach.

#### 7.1.3.1. Ultimate Limit State

Table 11 resumes the results in the base of the cantilever, which is the only relevant point in the analysis.

In this element is important to select a rebar grid that respect the minimal mesh established on ACI-350 [12] (15,24 [cm$^2$/m]).

*Table 11 – ULS analysis for the structural wall*

| Wall | $M_{sd}$ $[kNm/m]$ | $\mu$ $[-]$ | $A_s$ $[-]$ | $[cm^2/m]$ |
|---|---|---|---|---|
| Band II | -10 068 | 0,139 | ϕ32//0,10 + ϕ32//0,10 (2 layers) | 160,64 |

The structural wall has also subjected to shear force resistance verification. In this test, it was considered an axial force equivalent to the self-weight of the element for both positions.

This test was performed for the base of the cantilever and for the position where the reinforcement passes from two to one layer. In the both case it verifies the criteria imposed.

#### 7.1.3.2. Serviceability Limit State

NP EN 1992-3 [13] presents a set of requirements that established a crack limitation with tightness needs. In this structure should be used a class 1 limitation which is equivalent to 0,15 [mm], for this structure.

*Table 12 – SLS analysis for the structural wall*

| Wall | $M_{ELS}$ $[kN.m]$ | $N_{ELS}$ $[kN.m]$ | $\sigma_c$ $[MPa]$ | $\sigma_s$ $[MPa]$ | $w_k$ $[mm]$ |
|---|---|---|---|---|---|
| Band II | -6712 | -1507 | -13,84 | 259,27 | 0,19 |

The previous results confirm all the verifications, except the crack width limitation. However, for the part of the structural wall that is being analysed it is not a relevant problem because it is not exposed to water like other parts are. Once it verified the general criteria (0,30 [mm]), it can be acceptable as positive result.

### 7.2. Finite Element Method

In this project, it is also performed a FEM analysis of the structure behaviour in SAP2000® [A]. This study provides, theoretically, better results because it considers the real boundary conditions and the deformability of the elements.

In the next subchapters, it is repeated the verifications to ULS and SLS for all the elements, so that we can compare the concrete reinforcements and the resultant bending moments and shear forces.

### 7.2.1. Roof beams

In this evaluation, it is considered the same geometric dimensions and strength classes of concrete than before, but this time it was designed all the beams to check the real needs in each one. For the reinforcement schemes, it is important to understand the impact of using precast concrete beams have on the reinforcement detail drawings and construction processes.

#### 7.2.1.1. Ultimate Limit State

According to the results, ULS requirements impose a reinforcement scheme that is incompatible with ELS. This way, it was necessary to change it to be suitable for both limit states.

In this evaluation, it was also designed the four corbels in beams B1 and B5, which are structural pieces where the secondary beams are assembled. These elements were designed using a strut-and-tie model (Figure 2).

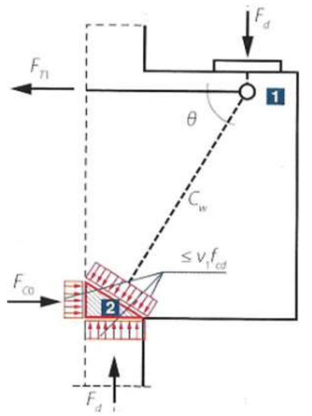


*Figure 2 – Strut-and-tie model* [14]

The failure analysis requires the introduction of a corbel reinforcement equals to 8 [$cm^2$]. This area should be distributed according to the norm NP EN 1992-1-1 [2].

#### 7.2.1.2. Serviceability Limit State

Regarding the main beams, the SLS is verified for all the beams with the second steel reinforcement schemes.

In this case, it is performed a study like the one explained before for the simplified models.

The results show maximum stresses for concrete and steel on beam B5. The bending moments is higher than cracking moment in all the cases and it reached the maximum width crack in some beams (B1, B5 and B6), but it never goes above the limit (0,30 [mm]).

The long-term deformation happens on beams B1 and B5 and it is equal to 12 [mm] which is a value within the limit (30 [mm]). The verification is also checked on secondary beams.

### 7.2.2. Technical slab

The model of the interior slab allows to create a local model that considers the boundary conditions and the deformability of the beams located interior limits of the panels studied previously.

In this case, it is no longer required to do an individual analysis to each panel.

This analysis follows the thickness defined before for the slab and dimensions of interior beams specified in the general arrangement drawings and resumed in Table 13.

*Table 13 – Geometric dimensions of the interior beams*

| Interior beams | Cross section | |
|---|---|---|
| | $h\ [m]$ | $b\ [m]$ |
| V1 | 1,00 | 0,60 |
| V2 | 0,75 | 0,60 |
| V3 | 0,75 | 0,60 |

#### 7.2.2.1. Ultimate Limit State

The analysis performed in FEM allow to keep the same minimum reinforcement grid ($\phi$12//0,20 5,65 [$cm^2/m$]), but it imposes some reinforcement in the perimeter of the slab (supports).

The verification towards the shear forced shows no need for stirrups or other reinforcements on the slab.

#### 7.2.2.2. Serviceability Limit State

Regarding to SLS, the slab presents a maximum crack width equals to 0,29 [mm], which is the section with highest bending moment. Also in this section, it is registered the maximum tension in the concrete and in the steel, 255,1 [MPa] and 11,5 [MPa], respectively.

According to MEF results the maximum long-term deformation is located in the mid span and its value is equal to 9,1 [mm], which is lower than the maximum, 24,8 [mm].

### 7.2.3. Interior beams

According to the results provided by MEF analysis to the beams, the cross sections of the them could be changed in order to obtain the reduced bending moment ratiosmore suitable for the real needs of the structure.

Despite the long span, the interior beams do not present very high moments. The cracking control (SLS) is the most demanding state for these set of elements.

### 7.2.4. Structural wall

In this study, it is followed the previous geometric dimensions and strength classes of concrete defined for simplified models.

This analysis allows to introduce in the model, the boundary conditions in the perimeter of the all wall, in which three of four supports are fixed.

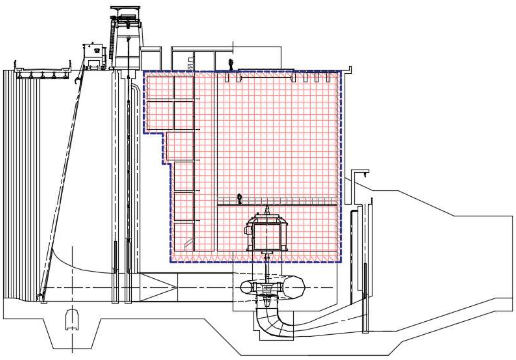

*Figure 3 – Structural wall model*

These circumstances allow to better distribute the load and decrease the bending moments values.

#### 7.2.4.1. Ultimate Limit State

Table 14 resumes the longitudinal reinforcement needed in both directions and surfaces.

It was also verified the shear force resistance is higher than designed values, even without considering axial force (self-weight).

*Table 14 – ULS analysis for the structural wall*

| Wall | Section | $M_{sd}$ $[kNm/m]$ | $\mu$ $[-]$ | $A_s$ $[-]$ | $[cm^2/m]$ |
|---|---|---|---|---|---|
| $m_{22}$ | Support (midpoint) | -2491 | 0,034 | ϕ20//0,20 +ϕ20//0,20 | 31,42 |
| | Support (quarter) | -1529 | 0,021 | ϕ20//0,20 +ϕ12//0,20 | 21,36 |
| | Span | 912 | 0,013 | ϕ20//0,20 | 15,71 |
| $m_{11}$ | Support (tailwater) | -1381 | 0,019 | ϕ20//0,20 +ϕ12//0,20 | 21,36 |
| | Support (headwater) | -2563 | 0,035 | ϕ20//0,20 +ϕ20//0,20 | 31,42 |
| | Span | 919 | 0,013 | ϕ20//0,20 | 15,71 |

#### 7.2.4.2. Serviceability Limit State

Throughout the gala analysis, it was possible to check that the initial reinforcement scheme was not suitable enough for SLS requirements, so it was necessary to change it for the horizontal direction according to Table 15.

*Table 15 – Steel reinforcement changes*

| Section | Reinforcement ELU | Reinforcement ELS |
|---|---|---|
| Support (headwater) | ϕ20//0,20+ϕ12//0,20 (21,36 cm²) | ϕ20//0,20+ϕ16//0,20 (25,76 cm²) |
| Support (tailwater) | ϕ20//0,20+ϕ20//0,20 (31,42 cm²) | ϕ20//0,20+ϕ20//0,20 (31,42 cm²) |
| Span | ϕ20//0,20 (15,71 cm²) | ϕ20//0,20+ϕ20//0,20 (31,42 cm²) |

After these changes, the structures had verified all the SLS requirements as it is shown for the relevant section in Table 16 and Table 17.

*Table 16 – ULS analysis for the wall in direction 11*

| Section | $N_{pp}$ $[kN/m]$ | $M_{ELU,11}$ $[kNm/m]$ | $M_{ELS,11}$ $[kNm/m]$ | $wk_{11}$ $[-]$ | $\sigma_{s,11}$ $[MPa]$ | $\sigma_{c,11}$ $[MPa]$ |
|---|---|---|---|---|---|---|
| Span | $[-]$ | 759 | 507 | 0,10 | 88,4 | 2,1 |
| Support (headwater) | $[-]$ | -2108 | -1406 | 0,30 | 245,1 | 5,8 |

*Table 17 – ULS analysis for the wall in direction 22*

| Section | $N_{pp}$ $[kN/m]$ | $M_{ELU,22}$ $[kNm/m]$ | $M_{ELS,22}$ $[kNm/m]$ | $wk_{22}$ $[-]$ | $\sigma_{s,22}$ $[MPa]$ | $\sigma_{c,22}$ $[MPa]$ |
|---|---|---|---|---|---|---|
| Span | -1457 | -1640 | -1094 | 0,02 | 19,6 | 3,08 |
| Support | -1505 | -2379 | -1576 | 0,07 | 70,2 | 5,34 |

Lastly, it was calculated the maximum long-term deformation of this element (3,4 [mm]), which is less than the limit (42,1 [mm]).

## 8. Conclusion

This dissertation consisted on the evaluation of one of the alternatives structures for a hydroelectric project located in Mozambique. The analysis combined the safety check of the overall stability of the structure and the internal stability of a set of relevant elements, through simplified models and numerical models, and it includes the presentation of the general layouts and the detailed reinforcement drawings.

As a result of the dissertation, it was possible to understand the consequences that regulations have on the design, not only into the geometry and mass of the power plant for the global stability, but also the relation between the structure and environment around. The exposure conditions present different challenges that must be considered in the material selection for the internal stability validation.

From the technical point of view, overturning is the scenario which has a safety coefficient closest to its limit. However, it is two times bigger than the calculated value. This result allows us to conclude that it is possible to redefine some concrete elements, which would reduce the overall cost of the project.

In addition, it is also important to ensure the stresses criteria is verified, because it is more conservative criteria for the overall stability. According with the obtained results the foundation is fully compressed for all the evaluated scenarios, so it is not expecting to have traction nor cracking on it.

To sum up, this study shows the importance the structure weight has in the static equilibrium, and why it is relevant to work with an accurate value. Also important is the reduction the drain curtain imposes on uplift pressure distribution, which helps to ensure structural adequacy.

Regarding the structure internal stability, cracking is the most determinant phenomenon in SLS, and it is responsible for the redefinition of reinforcements in many sections. Moreover, the norm NP EN 1992-1-1 [2] does not propose any requirements related with the minimum reinforcement for mass concrete elements nor for specific elements in dams. This situation is analysed in other references ([6], [8], [12]).

In the final phase of this dissertation, it was carry out a comparative analysis between the two models used. This allow to verify the simple models present a convergence of results for the majority of cases, and proved to be a reliable and practical first approach. Only in the case of the structural wall, the different in the boundary conditions impose high difference in values of bending moment and shear forces, and consequently different rebar reinforcements.

In conclusion, it would be interesting to develop a global three-dimensional model (considering all geological conditions and load) in order to obtain more precise results and allow to perform an optimization between the reinforcement and dimensions of the elements. This model would also allow to consider cracking phenomena due to shrinkage and the effects of ambient temperature variation.

Finally, it would be interesting to develop a research work, focused on mass concrete and its structural design, namely minimal amounts of reinforcement. These aspects are not entirely treated in NP EN 1992-1-1 [2], concerning special structures such as dams and mass concrete elements.

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