STRUCTURAL DESIGN OF A WATER INTAKE IN REINFORCED CONCRETE

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ABSTRACT

The main objective of this dissertation is to analyse and to verify the global and the internal stability of a structural block of a water intake in reinforced concrete, being designed the reinforcement of some structural elements, in accordance with the Portuguese and international legislation in force.

In a first phase, a three-dimensional geometric model is created for a better understanding and to obtain some data necessary for further analysis.

In a second phase, are defined the structural materials and are quantified the acting actions on the structure. In order to analyse the global safety of the structure are calculated the global safety factors.

In a third phase, to design some structural elements are verified the safety for the ultimate limit state (ULS) and the safety for the serviceability limit state (SLS), using simplified models, which allows to get a first approach of the quantities of reinforcement required, and also a three-dimensional finite element model in order to compare and to complement results.

Lastly, are presented the detailed general arrangement drawings and the reinforcement drawings of the structural elements analysed.

KEYWORDS: Hydraulic Structures, Water Intake, Reinforced Concrete, Stability, Structural Design, Three-Dimensional Modelling.

1. INTRODUCTION

1.1. MAIN GOALS

The aim of this work is to evaluate the global and internal stability of a structural block of the water intake of Cambambe Hydroelectric Power Plant 2 in reinforced concrete, related with. the project of power strengthening of Cambambe hydroelectric-installation.

The geometric definition of the structure is presented in general arrangement drawings. As the blocks are separated by contraction joints, all these blocks are structurally independent.

In order to understand the complex geometry of the structure, a three-dimensional geometric model was created, not only for a better understanding but also to obtain some data necessary for the analysis, such as the weight of the concrete structure, the weight of the earthfill and also to quantify the uplift resultant forces.

The global safety of the structural block is assured with the calculation of safety factors for: (i) sliding, (ii) uplift, (iii) overturning and (iv) stresses in the foundation for each design situation.

It is evaluated the internal stability, verifying the safety for the ultimate limit state (ULS) and serviceability limit state (SLS).

In this dissertation are presented the design of the following elements: (i) foundation slab, (ii) slab over the hydraulic tunnel, (iii) supporting beam of the servomotor and (iv) structure supporting the crane rail

The design of these elements is based on simple models and based on a three-dimensional finite element model (*FEM*) which were implemented on *SAP2000* software.

The main amounts of reinforcement are calculated and detailed reinforcement drawings are presented for the above-mentioned elements

1.2. BACKGROUND

The hydroelectric installation is located in Kwanza river, near the city of Dondo, approximately 180 km southwest from the city of Luanda. It is constituted by an 88 m height arch dam which the main purpose is to produce hydro energy.

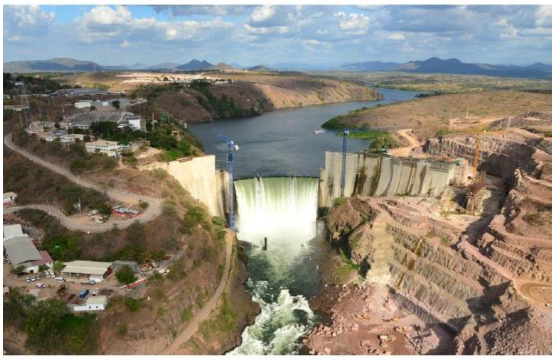

Figure 1 – Hidroelectric power facility of Cambambe [1]

The Cambambe dam and Cambambe Hydropower plant 1, with an installed capacity of 180 MW, were built between 1959 and 1963.

In 2009, started a project of rehabilitation, update and enlargement of Cambambe hydroelectric facility. The height of the dam was raised 20 m and the full level of water storage was increased 28 m, which increased the total capacity of the reservoir from 28,7x10^6 m^3 to 50x10^6 m^3. At the same time, to increase the energy production, the turbines of Cambambe 1 were changed for four turbines with an installed capacity of 260 MW. In Cambambe 2 were installed four Francis turbines with an installed capacity of 700 MW. [2] [3]

2. 3D MODELLING

In figure 2, is shown a three-dimensional model made in the *AutoCAD* software, in accordance with the information presented in the general arrangement drawings.

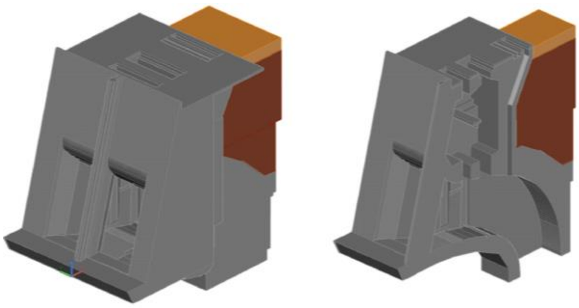

Figure 2 – Three-dimensional model

Through this software is obtained the position of the centre of gravity of the structure and also volumes. The volume of concrete is 5579 m^3 and the volume of the earth fill is 1993 m^3.

3. STRUCTURAL MATERIALS

The durability is a concept to take into account during the design of the structure. According to NP EN 1990 [4], structures should verify resistance and serviceability requirements, during its working life, without unforeseen maintenance or unforeseen costs.

Due to the importance of this structure, according to NP EN 1992-1-1 [5] the working life of it is 100 years and therefore it corresponds to structural class S6. The exposition class considered is XC4 and for this class, the NP EN 1992-1-1 [5] defines a minimum concrete cover of 50 mm.

The materials selected are the structural steel S500, concrete C30/37 for the structure and C16/20 for regularization layers.

According to the norm NP EN 206-1 [6], the concrete specifications results in:

C30/37; XC4(Pt); Cl 0,40; D_{max} 32; S3

4. DESIGN SITUATIONS AND COMBINATIONS OF ACTIONS

In accordance with the Portuguese Regulation on Dams Safety (RSB), the design of these structures must verify the safety for (i) current scenarios and (ii) failure scenarios. [7]

Additionally, according with other references [8], should be checked also a construction scenario and a limit scenario.

4.1. DESIGN SITUATIONS

The **construction scenario** (**Scenario 1**) corresponds to the conditioning situation during the constructive phase in which the structure is complete with the entire fill placed on the back part. It is also considered an overload applied on the top of the fill and the action due to the compaction of the soil.

The **usual scenarios** are described by the combination of actions that occur with a high probability during the working life of structures. Are considered three different current scenarios that may occur during the working life of this structure:

Scenario 2 – Scenario of operation. It represents the situation of operation of the water intake, in which the water are considered at full level of storage both in the front and back of the structure acting along with the action of the fill and a live load on the top of it.

Scenario 3 – Scenario of maintenance considering the closure gate and the maintenance gate are closed, therefore no water inside of the structure. In this situation is take into account the actions due to the water and due to the fill presented on the back of the structure and also a live load applied on the top of it.

Scenario 4 – Scenario seismic in which is considered the same situation described in Scenario 3, adding a project seismic load. In comparison to scenario 2, this scenario is the conditioning situation because there is no water inside the chamber (favourable for the global stability of the structure)

The **failure scenarios** correspond to a combination of actions with a low probability of occurrence during the working life. Are considered in this dissertation the following situations:

Scenario 5 – Seismic scenario in which is considered the same situation described in Scenario 3, adding a maximum seismic load.

Scenario 6 – Seismic scenario during construction phase. This situation represents when a maximum seismic load happens during the construction phase.

4.2. GLOBAL STABILITY

The limit states assessed in order to check the global stability of this structure are:

- EQU - Loss of equilibrium of the structure due to **overturning stability** and loss of equilibrium due to **sliding**;
- UPL - Loss of equilibrium of the structure due to uplift;
- GEO - Failure or **excessive deformation** of the ground.

In accordance with the Portuguese legislation presented in [7] and other international regulations [8], the structure in analysis must check the following minimum safety factor shown in table 1:

Table 1 - Safety factors for global stability

Scenario	$SFS_{\emptyset,min}$	SFF_{min}	FSO_{min}	FST_{min}
S1	1,3	1,2	1,3	2,0
S2–S4	1,5	1,3	1,5	3,0
S5–S6	1,2	1,1	1,2	1,5

4.3. INTERNAL STABILITY

According to NP EN 1990 [4], the structure must be designed to resist to ultimate limit states (ULS) and serviceability limit states (SLS). The first are related with the safety of people and of the structure, while the second are related to the use/operation of the structure.

4.3.1. ULTIMATE LIMIT STATES (ULS)

The ultimate limit state of all elements must be evaluated, according to the equation below:

$$E_d \leq R_d \qquad (1)$$

in which

$$E_d = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \sum_{i\geq 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

4.3.2. SERVICEABILITY LIMIT STATES (SLS)

The verification of SLS is related with several aspects, such as the normal functioning of the structure, its appearance and comfort to users. In order to satisfy this, the following equation must be checked:

$$E_d \leq C_d \qquad (2)$$

in which

$$E_d = \sum_{j\geq 1} G_{k,j} \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i\geq 1} \psi_{0,i} Q_{k,i}$$

4.3.3 PARTIAL SAFETY FACTORS

For the previous combinations, the loads are multiplied by the following load factors presented in table 2:

Table 2 – Partial safety factors

	ULS		SLS
Load	Adverse	Favourable	
Weight	1,35	1,00	1,00
Soil	1,50	1,00	1,00
Live Load	1,50	0,00	1,00
Water	1,50	1,00	1,00

5. ACTIONS

The following actions are considered: (i) self-weight of the structure; (ii) other permanent loads, (iii) live loads; (iv) soil lateral pressures, (v) weight of the earthfill, (vi) weight of water, (vii) hydrostatic pressure, (viii) uplift pressure, (ix) seismic actions.

6. GLOBAL STABILITY

The global stability of the structure is based on the hypotheses that the structure has rigid body behaviour, therefore the following verifications have to be satisfied:

6.1. SLIDING

The verification against failure by sliding is given by the following equation:

$$\frac{(\sum V - U) \times tg(\emptyset)}{\sum H} = SFS_{\emptyset} \geq SFS_{\emptyset,min} \qquad (3)$$

For each scenario are shown in the table 3 and table 4, respectively, the forces values and the quantification of the safety factor.

The contribution of cohesion is neglected.

Table 3 – Forces considered for verification against failure by sliding

Scenario	$\sum V$ $[kN]$	U $[kN]$	$\sum H$ $[kN]$
S1	167335	-	15172
S2	231228	-126743	8713
S3	210270	-126743	8713
S4	209009	-126743	16880
S5	208379	-126743	20964
S6	165226	-	25716

Table 4 -Safety factors against failure by sliding

Scenario	$SFS_{\emptyset}$	$SFS_{\emptyset,min}$
S1	8,48	1,30
S2	7,99	1,50
S3	6,39	
S4	4,43	
S5	3,54	1,20
S6	5,84	

6.2. UPLIFT

The safety of the structure against failure by uplift is assured if the following equation is verified:

$$\frac{\sum V}{U} = SFF \geq SFF_{min} \qquad (4)$$

The quantification of $\sum V$ and U are the same represented in table 3.

The safety factor against failure by uplift are shown in the next table:

Table 5 – Safety factors against failure by uplift

Scenario	SFF	SFF_{min}
S1	-	1,20
S2	1,82	1,30
S3	1,66	
S4	1,65	
S5	1,64	1,10
S6	-	

6.3. OVERTURNING STABILITY

Verification against failure by toppling is given by the following equation:

$$\left|\frac{\sum M_{stb}}{\sum M_{dst}}\right| = SFO \geq SFO_{min} \qquad (5)$$

In the table below it is shown the result moments applied on the structure.

Table 6 – Quantification of moments

Scenario	$\sum M_{stb}$ $[kN.m]$	$\sum M_{dst}$ $[kN.m]$
S1	2874582	-185279
S2	3536014	-1869084
S3	3159193	-1869084
S4	3159193	-1971282
S5	3159193	-2022382
S6	2874582	-332827

The safety factors against toppling failure are presented in table 7:

Table 7 – Safety factors against failure by toppling

Scenario	SFO	SFO_{min}
S1	15,51	1,30
S2	1,89	1,50
S3	1,69	
S4	1,60	
S5	1,56	1,20
S6	8,64	

6.4. STRESSES IN THE FOUNDATION

In order to check the safety, the following equation must be satisfied:

$$\sigma_{max} \leq \sigma_{adm} \qquad (6)$$

The foundation soil is composed by siltstone and conglomerate rocks with an admissible compression resistance equals to 1,41 MPa.

In the table below are presented the stresses on the foundation for each scenario. The conditioning vertical seismic direction is downwards. The stresses are calculated using the following equation:

$$\sigma = \frac{N}{A} \pm \frac{M}{W} \qquad (7)$$

Table 8 – Stresses verification

Scenario	$\sigma_{upstream}$ $[MPa]$	$\sigma_{downstream}$ $[MPa]$	SFT	SFT_{min}
S1	0,258	0,229	5,46	2,0
S2	0,168	0,140	8,38	3,0
S3	0,148	0,100	9,55	
S4	0,171	0,081	8,25	
S5	0,183	0,072	7,72	1,5
S6	0,290	0,204	4,86	

For all the static scenarios, the values of the stresses obtained are compression stresses.

7. INTERNAL STABILITY

7.1. ULTIMATE LIMIT STATES (ULS)

In this section are evaluated the simple models for the beam on the top of the water intake and it is also made a manual calculation of strut and tie analysis.

7.1.1. BEAM

This beam is located on the top of the water intake structure. According to Table 4.4N of NP EN 1992-1-1 [5], the concrete cover is 50 mm once it is exposed to exposure class $XC4$. The span considered is 10,33 m. On this beam are applied a distributed load (p_{sd}) equals to 188 kN/m and a concentrated load (P_{sd}), at the mid span, equals to 1015 kN which is related to the lifting of the maintenance gate.

The bending reinforcement must be within a range of a minimum and maximum steel area, being the minimum reinforcement given by, equation 8:

$$A_{s,min} = 0{,}26 \times \frac{f_{ctm}}{f_{yk}} \times b \times d \qquad (8)$$

The minimum shear reinforcement used is given by:

$$\left(\frac{A_{sw}}{s}\right)_{min} = 0{,}08 \times \frac{\sqrt{f_{ck}}}{f_{yk}} \times b \qquad (9)$$

In table 9 and 10 are represented the reinforcement needed for the design of the beam.

Table 9 – Longitudinal reinforcement needed in the beam

Section	M_{Ed} $[kN.m]$	μ $[-]$	ω $[-]$	A_s $[cm^2/m]$
Support	2978,3	0,03	0,03	41,02
Span	2144,3	0,02	0,02	41,02

Table 10 – Transversal reinforcement needed in the beam

Section	V_{Ed} $[kN]$	$V_{Ed,zcotg\theta}$ $[kN]$	$\left(\frac{A_s}{s}\right)$ $[cm^2/m]$
Support	1476,2	979,1	18,40
Span	509,0	-	18,40

7.1.2. DISCONTINUTY REGIONS

Due to its geometry or type of load applied on it, there are some zones in which its behaviour is different from Bernoulli's theory and therefore is needed to use other type of analyse. It is the case of the support of the servo-motor beam and the case of the support of the crane rail. On these cases are used a strut and tie model. According to Appleton [9], these models are valid since the compression tension of the concrete does not surpass the maximum compression tension.

7.1.2.1 SUPPORT OF SERVO-MOTOR BEAM

In the following table is presented the acting force applied in the beam and the equivalent forces applied on the ties and on the strut.

Table 11 - Strut and tie model for the support of servo-motor beam

$F_{d,max}$ $[kN]$	F_t $[kN]$	F_{cd} $[kN]$
1014,8	409,2	1094,1

In this case is required 9,54 cm^2/m, being adopted 6ϕ16 for a strip of 1,0 m width.

The compression on each node is presented in the following table:

Table 12 - Analysis of nodes

Node	σ_c $[MPa]$	σ_{Rd} $[MPa]$
1	2,89	14,96
2	1,45	17,60

7.1.2.1 SUPPORT OF CRANE RAIL

In the following table is presented the acting force applied in the beam and the equivalent forces applied on the ties and on the strut.

Table 13 - Strut and tie model for the support of crane rail

$F_{d,max}$ $[kN]$	F_t $[kN]$	F_{cd} $[kN]$
424,8	165,7	456,0

Although is just necessary 7,91 cm^2/m, as this section is very thick is adopted Φ20//0,20 (15,71 cm^2/m) which is the minimum shrinkage and temperature reinforcement, according to ACI-350-01. [10]

The compression on each node is presented in table 14:

Table 14 – Analysis of nodes

Node	σ_c $[MPa]$	σ_{Rd} $[MPa]$
1	0,41	14,96
2	0,21	17,60

7.2. SERVICEABILITY LIMIT STATES (SLS)

In this dissertation are evaluated (i) the deflection of the members, (ii) the stress of structural materials and (iii) the crack width.

The stress of the materials are limited, in serviceability, in order to ensure the steel is not yet in yielding and to ensure the concrete has no micro-cracks, therefore steel stress σ_s is limited to a maximum value of $0{,}80 f_{yk}$ and concrete compression σ_c is limited a maximum of $0{,}60 f_{ck}$.

The maximum crack width is 0,30 mm as recommended by NP EN 1992-1-1 [5] and EN 1992-3. [11]

For these SLS verifications are considered the characteristic combination.

8. FINITE ELEMENT MODEL

The FEM analysis of the structure gives more realistic data about the structure behaviour, once it has into account the real support conditions, the load paths, and the real stiffness of all elements among other considerations.

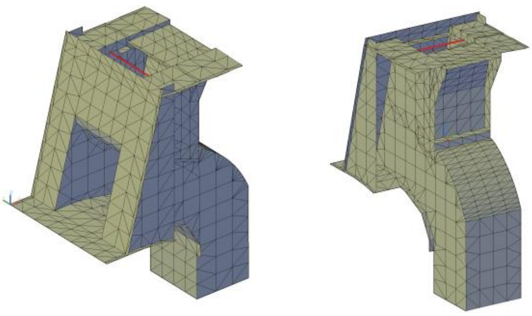

Figure 3 – Three dimensional FEM mesh

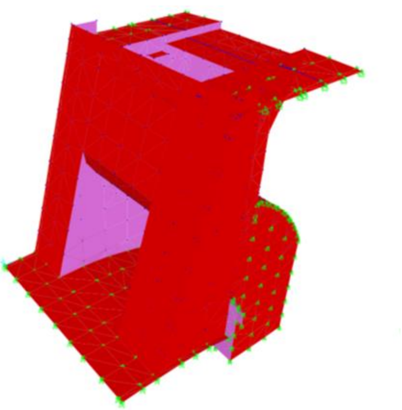

Figure 4 – Three dimensional FEM model

The present model is constituted by shell-thin elements (for slabs and walls) and frames elements (beams).

In order to take into account the real interaction soil-structure, the model is simulated with springs in the joints under the foundation slab and around the hydraulic tunnel. These springs just have compressive stiffness, therefore for a correct behaviour of it, is necessary to do a non-linear analysis of the structure.

The coefficient of soil reaction was calculated according to Vesic, given by equation (10), being equal to 132,32 MN/m^3. [12] [13]

$$k_s = 0{,}65 \sqrt[12]{\frac{E_s \times b^4}{E_f \times I_f}} \times \frac{E_s}{b(1-\upsilon_s^2)} \qquad (10)$$

8.1 ULTIMATE LIMITE STATE

8.1.1. FOUNDATION SLAB

The reinforcement of the foundation slab is shown in table 15 and 16. Were considered several regions and critical design sections, in order to optimize the amounts of required reinforcement. This slab is designed for bending with axial force.

Table 15 – Foundation slab reinforcement - direction 1

Section	Face	Reinforcement	A_s $[cm^2/m]$
A	Sup.	3 layers ϕ32//0,15	160,85
	Inf.	1st and 2nd layer ϕ32//0,15 3th layer ϕ25//0,15	139,96
B	Sup.	3 layers ϕ32//0,15	160,85
	Inf.	3 layers ϕ32//0,15	160,85
C	Sup.	3 layers ϕ32//0,15	160,85
	Inf.	1st and 2nd layer ϕ32//0,15 3th layer ϕ25//0,15	139,96
D	Sup.	3 layers ϕ32//0,15	160,85
	Inf.	3 layers ϕ32//0,15	160,85
E	Sup.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
	Inf.	3 layers ϕ32//0,15	160,85

Table 16- Mat foundation reinforcement - direction 2

Section	Face	Reinforcement	A_s $[cm^2/m]$
A	Sup.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
	Inf.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
B	Sup.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
	Inf.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
C	Sup.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
	Inf.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
D	Sup.	1st layer ϕ32//0,15 2nd and 3th layer ϕ25//0,15	119,07
	Inf.	3 layers ϕ25//0,15	98,17
E	Sup.	1st layer ϕ32//0,15 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	102,70
	Inf.	1st and 2nd layer ϕ25//0,15 3th layer ϕ25//0,30	81,81

It is necessary to provide transversal reinforcement, accordingly with the Portuguese legislation (REBAP). Therefore, is adopted the following transversal reinforcement (single legged stirrups) for the different stripes/sections:

Table 17 – Transversal reinforcement

Section	Transversal Reinforcement
B	Stirrups 1L. ϕ12//0,30 (0,30)
C	Stirrups 1L. ϕ12//0,30 (0,30)
D	Stirrups 1L. ϕ12//0,30 (0,30)
E	Stirrups 1L. ϕ20//0,30 (0,30)

For slab foundations is also necessary to check if the acting compression in the foundation is lower than the maximum compressive resistance. The reached acting compression is around 1 MPa while the maximum compressive resistance is 1,41 MPa, therefore the safety is checked.

8.1.2 SLAB UNDER THE EARTH FILL

Are shown in the table 18 and 19 the reinforcement used in this slab. It was design as well for bending with axial force.

Table 18 - Longitudinal reinforcement - direction 1

Section	Face	Reinforcement	A_s $[cm^2/m]$
A	Sup.	1st layer ϕ25//0,20 2nd and 3th layer ϕ20//0,20	55,96
	Inf.	1st layer ϕ32//0,20 2nd and 3th layer ϕ25//0,20	89,30
B	Sup.	1st layer ϕ25//0,20 2nd layer ϕ20//0,20	40,25
	Inf.	2 layers ϕ32//0,20	80,42
C	Sup.	2 layers ϕ20//0,20	31,42
	Inf.	1st layer ϕ32//0,20 2nd and 3th layer ϕ25//0,20	89,30
D	Sup.	3 layers ϕ20//0,20	47,12
	Inf.	1st layer ϕ32//0,20 2nd and 3th layer ϕ25//0,20	89,30

Table 19 - Longitudinal reinforcement - direction 2

Section	Face	Reinforcement	A_s $[cm^2/m]$
A	Sup.	1st and 2nd layer ϕ20//0,20 3th layer ϕ16//0,40	36,44
	Inf.	3 layers ϕ20//0,20	47,12
B	Sup.	1st layer ϕ20//0,20 2nd layer ϕ20//0,40	23,56
	Inf.	1st layer ϕ20//0,20 2nd layer ϕ20//0,40	23,56
C	Sup.	1st layer ϕ20//0,20 2nd layer ϕ20//0,40	23,56
	Inf.	1st and 2nd layer ϕ20//0,20 3th layer ϕ16//0,40	36,44
D	Sup.	3 layers ϕ20//0,20	47,12
	Inf.	1st and 2nd layer ϕ20//0,20 3th layer ϕ16//0,40	36,44

In order to check the safety verification for shear force in section D, is necessary to introduce single legged stirrups ϕ16//0,40 (0,40).

8.1.3 BEAM

Table 20 – Acting bending moment

Section	M_{Ed} $[kN.m]$	μ $[-]$	ω $[-]$	A_s $[cm^2/m]$
Support	2667,7	0,03	0,03	41,02
Span	2695,1	0,03	0,03	41,02

In comparison with the simple model values, the difference between these values are small, although the value of the bending moment on the support is lower than the value obtained through the simple model, but the *FEM* analysis considers the real stiffness of the elements, giving more realistic data.

As the servo-motor introduce a torsion moment T_{sd} to the beam equals to 476,9 $kN.m$, therefore the design of it should take this force into account too.

In table 21 and 22 are presented, respectively, the solution adopted for the longitudinal and transversal reinforcement.

Table 21 – Longitudinal Reinforcement

Section	Face	Reinforcement used	$A_{s,adopt}$ $[cm^2]$
Support	Sup.	2ϕ25 + 13ϕ20	50,66
	Inf.	2ϕ25 + 12ϕ20	47,52
Span	Sup.	2ϕ25 + 8ϕ20	34,95
	Inf.	2ϕ25 + 12ϕ20	47,52

Table 22 – Transversal Reinforcement

Outer Stirrups		Inner Stirrups	
Reinforcement	$\left(\frac{A_{sw}}{s}\right)$ $[cm^2/m]$	Reinforcement	$\left(\frac{A_{sw}}{s}\right)$ $[cm^2/m]$
2L. stirrups ϕ12//0,20	5,65	6L. stirrups ϕ10//0,20	23,58

To check the support and the mid-span cross section was used *GaLa Reinforcement* software. For these cross sections is obtained that the ratio between applied load and maximal permissible load is 0,264 for the support cross-section and 0,349 for the mid-span cross-section.

8.2 SERVICEABILITY LIMITE STATE

8.2.1 FOUNDATION SLAB

The maximum tension in concrete is 9,8 MPa and the maximum tension in steel is equal to 295,0 MPa, which are less than the maximum allowed tensions, respectively 18 MPa for concrete and 400 MPa for structural reinforcement, using indirect control of cracking.

The maximum crack width obtained by direct calculation is 0,30 mm which is the equal to the maximum value allowed.

8.2.2. SLAB UNDER THE EARTH FILL

The deformation of this slab is near 3 mm and the maximum allowed deformation is 21 mm.

On this slab is reached a maximum tension in concrete equals to 6,0 MPa and maximum tension in steel equals to 239,3 MPa.

The maximum crack width obtained is 0,28 mm, lower than the maximum allowed.

8.2.3. BEAM

The first aspect analysed is that the cracking bending moment of this cross section is higher than the acting bending moment, therefore there is no cracks on the beam. In spite of this, is normal in thick elements to consider that the section is cracked due to many reasons, such as the effect of the hardening of the concrete, the shrinkage among other aspects. [9]

The cross section was analysed in *GaLa Reinforcement* software assuming it is cracked and for the acting forces, the expect crack width obtained is 0,30 mm, which is equal to the maximum allowed by EC2.

In table 23 are shown the tensions in each material:

Table 23 – Tensions on concrete and steel

σ_c $[MPa]$	$\sigma_{c,máx}$ $[MPa]$	σ_s $[MPa]$	$\sigma_{s,máx}$ $[MPa]$
4,57	18	148,93	400

The long-term deformation expected is 15,3 mm which verifies the maximum of 20,7 mm.

10. CONCLUSION

In this project, the overturning of the structure represents the lowest safety factor of the global stability of the structure. It was verified for each static scenario the tension on the foundation is always compression.

In general, the thickness of some of these structural elements on hydraulic structures, and in the particular case of this project, the thickness of the slab under the earth fill and the beam, are conditioned by the global stability of the structure and not for the for the verifications of ULS or SLS.

Throughout this project, the crack width was the main problem for most the elements analysed. The thickness of the concrete cover conditioned the crack width verification, as a higher cover requires a more reinforcement area, while in other international legislation in force, it is possible to use less reinforcement areas.

The finite element model made in *SAP2000* was a crucial tool to evaluate the behaviour of this structure, even with the geometric simplifications that were made.

In a further phase of the project it would be interesting to proceed a more complex non-linear analysis taking into account the construction phases, an seismic analysis and to study the imposed deformations on the structure and cracking due to shrinkage and creep phenomenon, as it was just indirectly taken into account the minimum shrinkage and temperature reinforcement mentioned on ACI 350-01 [10].

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