STRUCTURAL DESIGN OF DIVERSION GALLERIES IN REINFORCED CONCRETE

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Abstract: Diversion galleries, as their name suggests, are means used to divert the flow of water towards a region away from a construction site. These provisional structures must correspond to a set of structural, as well as hydraulic and geotechnical safety standards. If any of the stipulated conditions fail, there is a high risk of flooding the nearby areas, which may cause unacceptable damages to any potential nearby populations.

The main goal of this dissertation is to verify the structural safety of a diversion gallery in reinforced concrete, located under a RCC (Roller Compacted Concrete) dam.

The main detail on these structures is the need to ensure (via reinforced concrete design) that the structure can support the load transmitted by the RCC.

Several data was provided beforehand, which include the dam's general geometrical information, materials used for its construction and the normal head water level of the dam.

This thesis is divided mainly in three parts. In the first part, a structural analysis is made on the gallery using tabular data for a similar structure. The second part consists of a structural analysis of the same gallery using a Finite Element Method with frame elements. The third and final part includes an analysis of the same structure with two dimensional shell elements, a comparison between results obtained for each model (with and without staged construction analysis) and a safety structural design for Ultimate Limit States and Serviceability Limit States.

1. Introduction

A diversion gallery can be defined as any physical means to divert the flow of water from a defined location. These types of structures range from small underground tubes to large concrete structures, but they must always have the appropriate geometry to ensure the hydraulic stability of the water that passes by.

A gallery can have several different geometrical configurations. In most cases, the choice of a configuration is indifferent, so the engineer has the free will to pick whichever one is desired.

Whenever any structure or foundation is projected to be built upon a watercourse, measurements must be taken before its construction, so as to avoid any contact between an unfinished structure and naturally flowing water. In these cases, it's common to build a diversion gallery and start the construction process as soon as the site is adequately dry.

This thesis focuses on structurally designing a series of diversion galleries that will serve as a base for a large RCC (Roller Compacted Concrete) dam. These galleries in particular will in no way create a "geometrical detour" to the flowing water, but will instead allow its free flow as the dam is being built on top of the galleries. Due to this, it's of utmost importance to carefully design its structural elements to ensure that it can safely withstand the weight of a large dam.

A large dam, like the one in this dissertation, requires very good foundation conditions, and a project for such a structure can be abandoned if the terrain underneath is unfit. Generally, conditions of lower quality than fractured bedrock are enough to be considered unacceptable for safety verification [1].

Roller compacted concrete is a type of concrete that has seen an increase in usage in the past decades. The process of construction involves its application, as if it were a regular concrete, followed by a compaction process using heavy machinery designed for this purpose. This process, however, must be thorough and done in layers. Joints between layers must also be carefully executed, as they are common weak points in the structure as a whole.

2. General work description

The entire structure will be a gravity dam sitting on top of a series of diversion galleries. These galleries will be pre-built and placed on top of the foundation terrain, with the eventual levelling layer of concrete. After the placement, joints are executed between the galleries, and finally, the application of RCC takes place on top of those. The layers of RCC become shorter as the building height increases, as shown in Figure 1. When the dam is completely built and ready for use, the galleries are sealed shut on the upstream side.

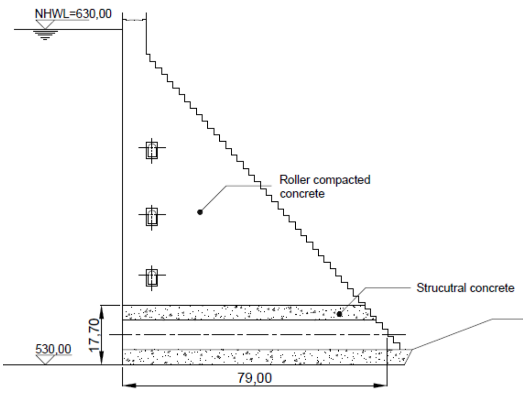

Figure 1 - Longitudinal cross-sectional cut of the structure

The dam is projected to hold back a stream of water flowing through a watercourse than can reach a height as high as 100 metres. Hence, the total height of the structure is 103 metres. Each gallery is 17,70 metres high and 24,00 metres wide, and has a specific geometrical definition, as shown in Figure 2. The total length of the galleries is approximately 79,00 metres.

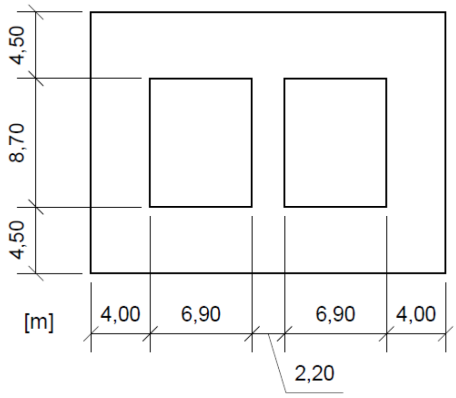

Figure 2 - Geometrical definition of the diversion gallery's cross-secion

The structure’s foundation is known to be a bedrock, which is suitable for its construction. But due to lack of information, little is known about this bedrock. So, in order to cover all possibilities, two geotechnical scenarios will be considered and analysed. In short, the bedrock can either be fractured or not. The difference in the behaviour between both cases is explained further ahead.

3. Project criteria: materials and actions

3.1. Materials

The materials to be used for this project were defined beforehand. For structural concrete, the class used was C25/30. The concrete to be applied on the RCC portion of the structure as well as for the levelling layer is only of class C12/15, since its structural behaviour is not as relevant.

Table 1 - Classes of concrete and their respective properties to be used in the project

Usage	Class	f_{ck} (MPa)	$E_{c,28}$ (GPa)
Structural Concrete	C25/30	25	31
RCC Concrete	C12/15	12	27

For the steel rebar, the class to be used was defined as A 500 NR.

Table 2 - Properties of the steel class used in the project

Steel Class	f_{yk} (MPa)	E_s (GPa)
A 500 NR	500	200

It is worth mentioning that, even though the class C25/30 was defined initially for this project, it does not comply with the terms specified in E 464 2005 [3]. These norms stipulate that a structure that is alternatively wet and dry during its lifetime belongs in the exposure class XC4, and therefore the lowest concrete class allowed by these terms is a C30/37. In spite of this, all calculations and models produced of this structure assumed that it consisted of a C25/30 concrete. Should the project move forward, all calculations and models are to be redone using the values correspondent to this new class.

In terms of concrete cover, it is important to ensure that the corrosion of reinforcement elements is to be avoided. As stated in the paragraph above, an exposure class of XC4 is regarded to be of high importance for the durability of a reinforced concrete structure.

According to [3], structures with an exposure class of XC4 and a lifetime of 100 years, have to ensure a concrete cover of 50 millimetres.

3.2. Actions

This structure will be designed mainly to verify safety criteria for Ultimate Limit States and Serviceability Limit States stipulated by [4]. The only exceptions are for safety regarding the shear force design, which will comply to the rules stated in REBAP (*Regulamento de Estruturas de Betão Armado e Pré-esforçado*) [5] and for the RCC dam design which will be executed as stipulated in EM 1110-2-2200 [6].

Three load combinations were considered to cover all possible worst case scenarios. However, there is a bigger emphasis in the constructive process of the dam. Table 3 and Table 4 include all safety coefficients and all partial coefficients, respectively.

Table 3 - Safety coefficients to be applied on the loads in ULS

ULS	PP	SP	IL
Combination 1	1,35	-	-
Combination 2	1,35	1,5	-
Combination 3	1,35	1,5	1,5

Table 4 - Partial coefficients to be applied on the loads in SLS

SLS	PP	SP	IL
Combination 1	1	-	-
Combination 2	1	1	-
Combination 3	1	1	1

3.2.1. Load combination 1

The first load case corresponds to a state wherein the dam is completely built and there are dry conditions. Therefore, the only weight applied on the gallery is the weight of the dam, as shown in Figure 3.

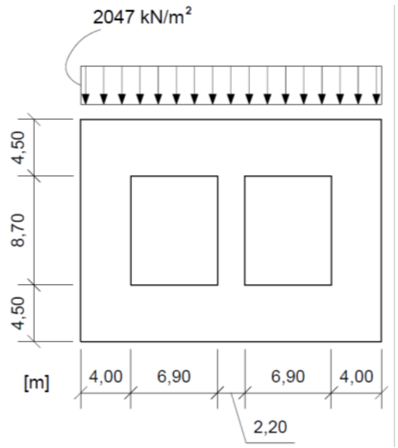

Figure 3 - Load combination 1

Since the dam is expected to reach a total height of 103 metres, the weight caused by the RCC portion (assuming γ_{RCC} = 24 kN/m^3) should be roughly equal to:

$$pp = 24 * (103 - 17{,}70) \approx 2047\ kN/m^2$$

3.2.2. Load combination 2

The second load combination represents a situation in which the normal head water level reaches its projected maximum of 100 metres, and causes, through infiltration, an upwards-facing pressure on the bottom surface of the gallery, as illustrated in Figure

4. This pressure, if considered being applied in a hydrostatic regime, is approximately:

$$sp = 100 * 9{,}8 = 980\ kN/m^2$$

Besides this load, it also includes the one caused by the weight of the dam, specified in 3.2.1.

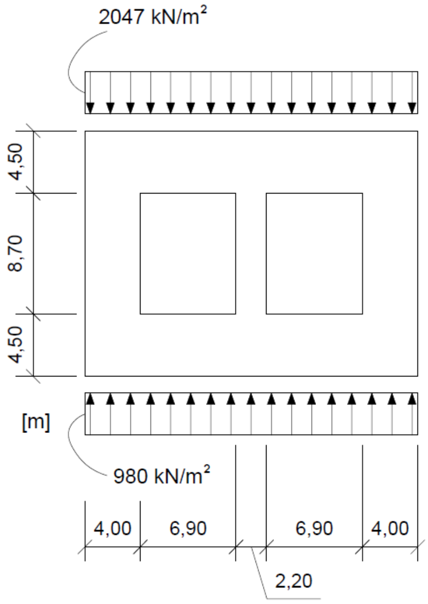

Figure 4 - Load combination 2

3.2.3. Load combination 3

The third load combination considers the possibility of a rupture in the joints between galleries, which can result in water infiltration in the gaps formed. This infiltration will cause lateral pressures on the sides of the galleries. Like the previous load combination, the scenario is worse when the water's level reaches its maximum of 100 metres. The lateral pressures on the gallery will be approximately:

$$Il_{top} = (100 - 17{,}70) * 9{,}8 = 806{,}5\ kN/m^2$$

$$Il_{bottom} = 100 * 9{,}8 = 980\ kN/m^2$$

Besides the lateral pressures, the buoyancy referred in 3.2.2 is also included, and so is the weight of the dam. The loads are represented in Figure 5.

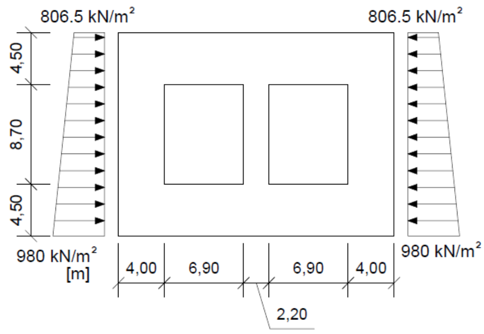

Figure 5 - Load combination 3

4. Modelling of the structure and safety verification

4.1. Model I

The first model to be used to calculate the structure's internal forces and moments consists of a series of one-dimensional elements. These create a structure on its own where its elements coincide with the axes of the elements of the real one, as shown in Figure 6.

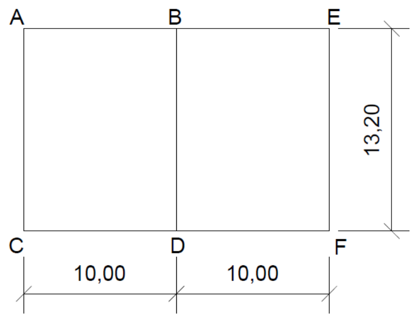

Figure 6 - Model of the gallery using one-dimensional elements

This model consists of using tabled values for certain loads. These values can be encountered in [7]. Since there is no load case that includes an upwards-facing pressure on the bottom surface, the only option is to consider this pressure as a "reduction" of the reaction caused by the weight of the dam. Conservatively, and in a safety point of view, it is not reasonable to consider a reduction in the terrain surface's reaction, and therefore, load combinations 1 and 2 are considered equal.

The tabled values (shown in Table 5) correspond solely to the moments on the nodes of the structure (A to F). All the other internal forces, including mid-span moments must be obtained through equilibrium, and are shown in Figure 7.

Table 5 - Moments obtained in the main nodes (values in kNm/m)

Section	Combinations 1 and 2	Combination 3
A; E	-5376.8	-20 653.5
B	-35692.9	-27 919.3
C; F	-5376.8	-20 833.8
D	-35692.9	-28 099.6

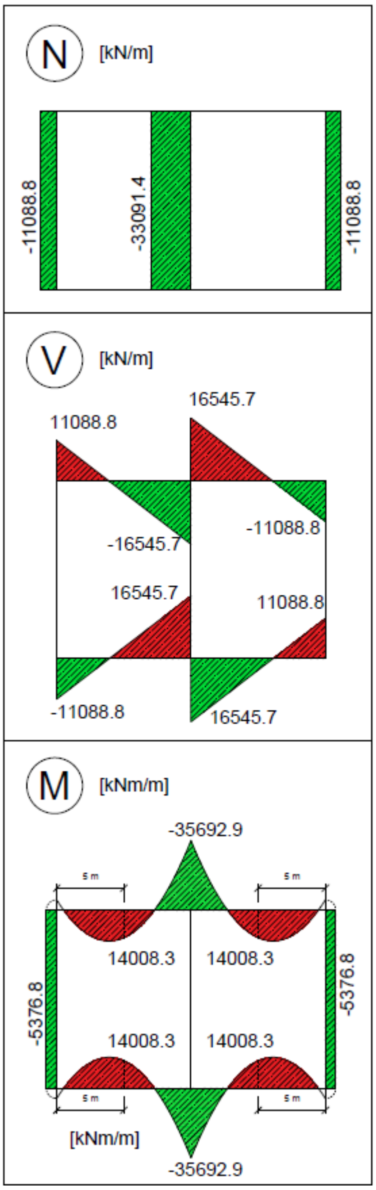

Figure 7 – ULS Internal forces and bending moment diagrams for the first (and second) load combination

Using these values, a complete structural design can be made with respect to Ultimate Limit States. However, this model is expected to generate results that do not adequately represent the reality. To have a notion of the amount of steel rebar necessary, the most influencing result would require one layer of ϕ32//0.100 plus three more layers of ϕ32//0.200 through sections B and D, for flexural strength alone.

Another reason to disregard this model's verisimilitude is present in the fact that this model did not take the elasticity of the foundation into account. The models described further ahead will take this into account.

4.2. Model II

Similar to Model I, the second will also consist of one-dimensional elements composing the entire structure. This one, however, will be executed using a finite element method with a structural analysis software program.

This model is expected to generate results that will be slightly more realistic that the previous one. The geometry is the same, but in this case it is possible to simulate the elastic support conditions.

4.2.1. Modelling the structure

It was referred that the foundation is necessarily a bedrock, albeit not knowing its condition. The most intuitive way to model the support conditions is to assume that the bottom parts of the gallery are in contact with a Winkler spring bed as shown in Figure 8, governed by the following equation:

$$R(x) = k_s \cdot w(x)$$

Where $R(x)$ is the reaction function for a certain position on the bar (x), k_s is the Winkler spring stiffness parameter, and $w(x)$ is the displacement function for the position x.

The Winkler spring coefficient depends on the type of foundation, and in general terms, gets higher as the stiffness of the foundation increases. Two values for the different scenarios were defined and are present in Table 6.

Table 6 - Case denomination and Winkler spring coefficients

Case F1	**Case F2**
Good geotechnical conditions	Reasonable geotechnical conditions
K = 7 500 000 kN/m³	K = 500 000 kN/m³

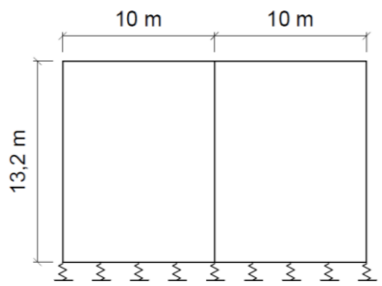

Figure 8 - Gallery Structure on a Winkler spring bed

Besides this, it is important to refer that since the foundation is no longer being considered rigid, there is now a difference between the first two loading scenarios. Since this is a foundation, the springs were modelled to resist to compression only.

Considering alternations between load cases and geotechnical scenarios, a total of six different cases are to be studied for the second model. They are summarized in Table 7.

Table 7 - Denominations for case scenarios to be analysed with Model II

Load combination	Geotechnical scenario	Case
1	F1	1
1	F2	2
2	F1	3
2	F2	4
3	F1	5
3	F2	6

Since this model is to be run in a software, a discretization must be made before the simulation is run. One-dimensional elements generally require low computational time, so finite elements as short as 0,125 metres long were adopted for the horizontal segments and 0,165 metres long for the vertical ones.

4.2.2. Results

The model was run for all six cases and all the internal force and moment (N, V, M) diagrams were obtained. A diagram for the spring bed reaction was obtained for every case as well.

The first analysis of the results will focus on the spring bed reaction, and a comparison between geotechnical scenarios will be made. To avoid an exhaustive presentation, only the reaction diagrams for the first load combination will be presented. They are shown in Figure 9 and Figure 10, (scenario F1 and F2, respectively).

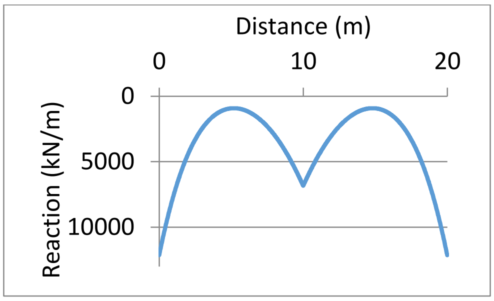

Figure 9 - Reaction diagram for load case 1 (K = 7 500 000 kN/m³)

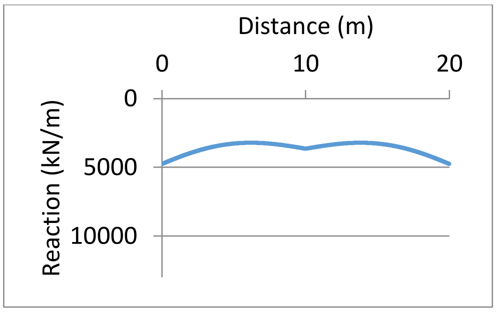

Figure 10 - Reaction diagram for load case 1 (K = 500 000 kN/m³)

By observing the reaction diagrams for each geotechnical case, it can be concluded that the diagram will have a bigger variability if the Winkler spring coefficient is larger. This means that the reaction gets closer to a constant diagram, the smaller this coefficient is. As a consequence, the shear force along these elements will be closer to a first degree function. It's also relevant to mention that, in the other load cases, the uplift due to the presence of water happens. This results in an overall smaller reaction, which generates zones where the reaction even reaches zero.

As for the internal force and moment diagrams, only the ones correspondent to Case 1 (Load combination 1 and geotechnical case F1) will be shown in Figure 11, Figure 12 and Figure 13 (N, V and M, respectively). This is, again, to avoid an exhaustive exposure in this document.

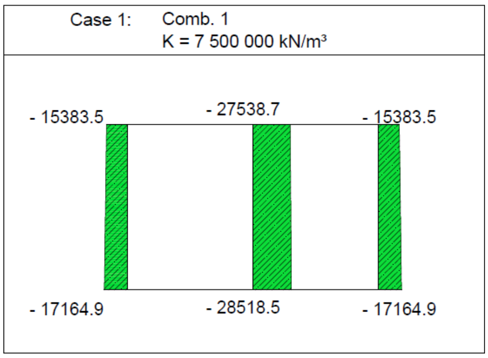

Figure 11 - Axial force diagram for case 1 (units in kN/m)

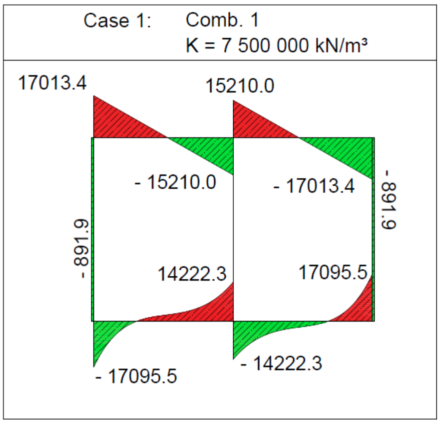

Figure 12 - Shear force diagram for case 1 (units in kN/m)

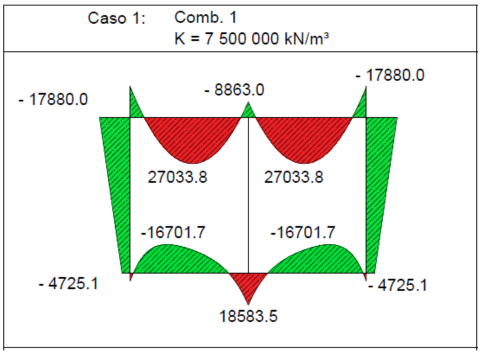

Figure 13 - Bending moment diagram for case 1 (units in kNm/m)

By analysing the diagrams above, and for every other case, there is a slightly noticeable reduction in negative bending moments and an increase in positive bending moments. Also, there is a small increase in axial force along the depth of the vertical segments due to the accountability of the gallery's own weight (which was neglected beforehand). The shear force diagrams also demonstrate how an elastic support can severely change their course.

Overall, the stresses obtained lightly reduce the amount of structural steel needed, to four layers of ϕ32//0.200 rebar.

4.3. Model III

The third model consists of the structure composed of two dimensional (shell) finite elements. Only the first load combination will be considered, though, as it is the only one that is relevant for the constructive process. Due to this, the direct results will no longer be expressed in internal forces and moments, but will instead be in stresses (force per area).

4.3.1. Modelling the structure

The structure will consist of the same material properties as the previous one, with the same geometry as the real gallery. Seven arbitrary sections (S1 to S7) were defined for obtaining the internal forces and moments indirectly through the shell stresses, and are represented in Figure 14.

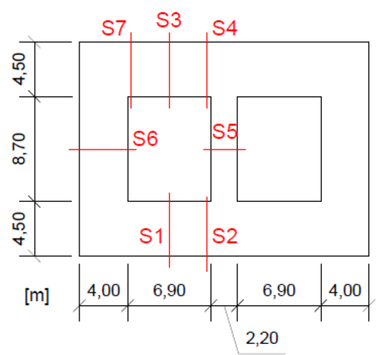

Figure 14 - Sections S1 to S7

Unlike the previous model, the discretization of a structure composed of shell elements must be carefully made, so as to avoid a low quality solution and a time-consuming process, simultaneously. The mesh used in this particular case was one that consists of ten elements along the thickness of the segments (excluding the vertical centre one, with six elements), and each one has four nodes.

4.3.2. Results

As mentioned earlier, the results for this simulation will be in the format of shell stresses. They are denominated as σ_{11} (horizontal axial stresses), σ_{22} (vertical axial stresses) and σ_{12} (shear stresses). They are represented in Figure 15, Figure 16 and Figure 17, respectively.

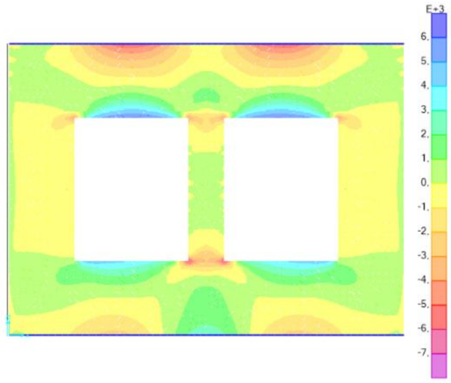

Figure 15 - σ_{11} stresses (model III, units in kPa)

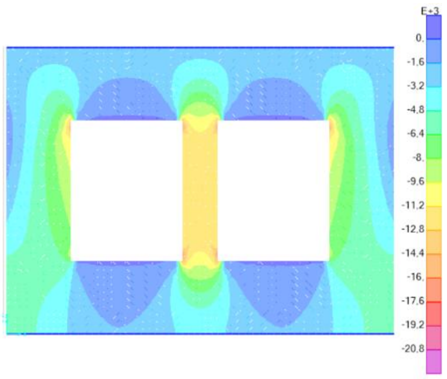

Figure 16 - σ_{22} stresses (model III, units in kPa)

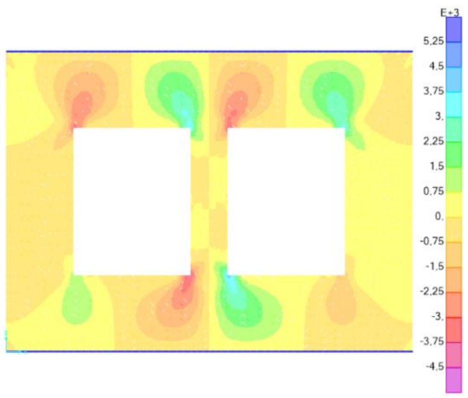

Figure 17 - σ_{12} stresses (model III, units in kPa)

By observing the stresses, the following conclusions can be made:

- there is a significant mid-span moment generated on the horizontal segments (σ_{11})
- there is a highly compressed area on the vertical segments (σ_{22})
- there are high concentrations of shear stresses near the vertical segments, suggesting higher values of shear force (σ_{12})

4.4. Model IV

The final model is the most important one for this dissertation, as it is the one that represents a situation as close to reality as possible. It consists of the same structure modelled with the same elements, but it also includes the constructive process for all the (2-metre-high) RCC layers.

4.4.1. Modelling the structure

Since this model actually takes the RCC into account as a material, and not just a load, it needs to be modelled as such, so it is necessary to know its physical properties. According to [8], formula 4-2, the modulus of elasticity (E, in psi) for the RCC, in the absence of experimental data, is given by:

$$E = 57000(f_{ck})^{1/2}$$

Where f_{ck} is the characteristic compressive strength of the concrete class used for the RCC, and must be expressed in psi. Using f_{ck} = 1740 psi (12 MPa), we get E = 2 377 658,5 psi (16,40 GPa).

According to paragraph 4.3 b. of the same document, the coefficient of Poisson for an RCC ranges between 0,17 and 0,22. And in the absence of empirical data, a value of 0,20 is recommended.

The mesh used in the model is identical to the one in Model III. As for the RCC mesh, its elements are as identical as possible to the ones on the gallery, each also composed of four nodes. The border between the reinforced concrete (C25/30) and the RCC has no incompatible nodes whatsoever.

4.4.2. Results

The final results obtained for this model will be used to calculate internal forces and moments and subsequently to design the section for safety checking. The stresses were calculated by the software and are represented in Figure 18, Figure 19 and Figure 20.

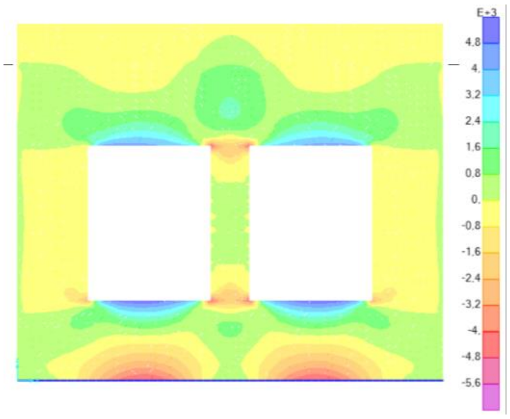

Figure 18 - σ_{11} stresses (model IV, K = 500 000 kN/m^3, units in kPa)

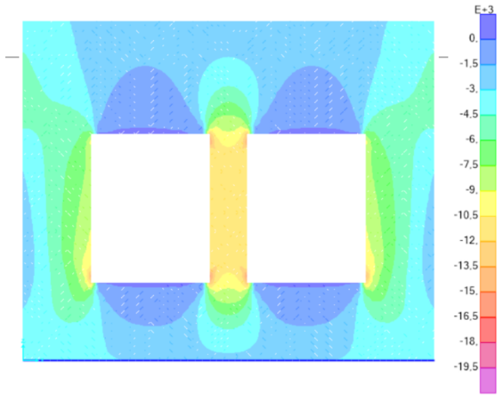

Figure 19 – σ_{22} stresses (model IV, K = 500 000 kN/m^3, units in kPa)

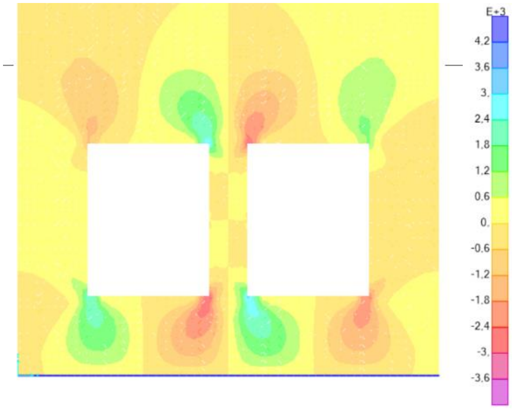

Figure 20 – σ_{12} stresses (model IV, K = 500 000 kN/m^3, units in kPa)

When compared to the previous model, the diagrams barely show any difference. The values, however, differ slightly, especially in the middle section between the two top segments. This happens due to the presence of the RCC on top of the gallery.

4.5. Safety check

Using only the values obtained for model IV, it is possible to create an envelope for internal forces and moments in order to verify the safety in the design of the structure. To obtain these internal forces and moments, the stresses must be integrated for every section (S1 to S7). For ultimate limit states, the envelope is represented in Table 8.

Table 8 - Envelope for the ULS internal forces and moments for sections S1 to S7 (model IV)

Section	Internal forces and moments		
	N_{sd} (kN/m)	V_{sd} (kN/m)	M_{sd} (kNm/m)
S1	-881,8	-1367,3	-15916,8
S2	268,4	7388,8	6180,6
S3	4423,0	-1448,2	8067,8
S4	4850,6	8412,2	-7322,2
S5	-19756,6	0,3	1952,5
S6	-19452,3	715,3	10220,4
S7	3175,1	6741,6	-2685,5

By observing the values in Table 8, in can be confirmed that the disparity in results between model IV and II is immense.

Besides ULS, the cracking check for SLS must also be executed. To summarize, the width of the crack in a service situation, w_k, must not be greater than the limit stipulated by [4] ($w_{k,lim}$), which is equal to 0,30 millimetres in this case. The results of the structural design can be checked in Table 9 and Table 10.

Table 9 - Reinforcement bars used for all sections (S1 to S7)

Sections	Top rebar	Bottom rebar
S1;S2	3 layers ϕ32//0.20	2 layers ϕ32//0.20
S3;S7	3 layers ϕ32//0.20	4 layers ϕ32//0.20
S4	4 layers ϕ32//0.20	4 layers ϕ32//0.20
S5	ϕ32//0.20	ϕ32//0.20
S6	2 layers ϕ32//0.20	2 layers ϕ32//0.20

Table 10 - Resistant bending moments and crack widths after concrete design

Section	M_{rd}^{+} (kNm/m)	M_{rd}^{-} (kNm/m)	w_k (mm)
S1	16377,4	-23485,2	0,28
S2	14008,8	-21174,8	0,20
S3	16202,2	-12584,5	0,23
S4	15309,8	-15309,6	0,25
S5	8583,2	-8583,2	0,00
S6	37743,9	-37743,9	0,00
S7	18798,0	-15187,9	0,20

As for shear design, the top sections will be constructed with ϕ32//0.20(//0.50) rebar and the bottom sections with ϕ32//0.20(//0.40). The vertical segments will not require shear reinforcement due to their high compression forces regardless of the situation.

Regarding the RCC, two safety checking criteria stated in [6] must be verified. For the load case considered, there cannot be any tension stresses in the structure and compressive stresses must not exceed 30% of the concrete class's characteristic compression strength (3,6 MPa).

The first criterion is roughly verified, however, the second is not entirely due to high vertical compression stresses on the sides, just above the gallery, which go as high as 4,7 MPa.

5. Conclusions

As a general rule, hydraulic structures are massive, and must be carefully designed knowing that the principles adopted in regular structures may not be reasonable in these cases. Besides this, it is also important to compare the order of magnitude in both stresses and the amount of steel reinforcement bars between common structures and hydraulic ones.

Regarding this case, one very important conclusion must be drawn about the overall design. Due to the high compressive stresses in the sides of the RCC block, a readjustment to the gallery's thickness would help reduce the stresses in the more fragile region.

Finally, it is worth mentioning that a project of this size and importance must include a very thorough hydraulic design as well. It should also include a check of the dam's global stability and a structural design of the plugs that will close the gallery after the construction of the dam. This, however is falls out of the context of this dissertation.

6. References

[1] Design of Gravity Dams, Bureau of Reclamation, 1976

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