Structural design of a rectangular non-elevated reservoir in reinforced concrete

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May 2016

Abstract

The following work has the purpose to perform the structural design of a water reservoir in reinforced concrete, according with the Portuguese and European regulations.

First, it is necessary to ensure that the structure will operate properly along the defined lifetime, with respect to the durability control of cracking.

After, the actions on the reservoir are quantified. The indirect ones are important, considering the time evolution along time of the concrete shrinkage and creep.

To perform the structural design, to the ultimate limit state and serviceability state, finite elements models have been developed.

It is also verified a structural behaviour for a seismic action. It is defined the response spectrum according with the NP EN 1998-1 [7]. The equivalent static forces of the seismic action are quantified as well for the finite elements model.

Finally, the structure is also designed to control the cracks and then it is verified the safety at ultimate state.

Key-words: Reservoir, structural design, reinforced concrete, cracking, seismic action

1 Introduction

Reservoirs are structures that contain fluids, in gaseous or liquid state. These could be made of reinforced concrete, prestressed concrete or steel, however the first ones are more common because of some important advantages such as the lowest cost of construction and maintenance.

Besides the material, reservoirs can be classified regarding the following dots: function, position, capacity, geometry, cover and tightness class.

The reservoir designed on this paper has the objective to supply water against a fire. On this way, the tank is non-elevated, especially because of the lower cost of construction. Besides that, a non-elevated tank has other advantages such as an easier operation as well a lower impact on the landscape view.

With a rectangular section base, the construction of the reservoir is easier and the axial forces are 10% lower comparing with a circular section.

2 Structural design rules for reservoirs

According with the section 2.1 of NP EN 1992-1-1, a concrete structure should accomplish some criteria, where commonly the most important is the structural strength. However, a reservoir has to ensure as well a good service behaviour, so it is given importance to durability and cracking control [5].

2.1 Durability

The Table 2.1 of NP EN 1990 defines that a common structure such as a reservoir should has a lifetime of 50 years. The same rule gives on section 2.4 aspects to take into account to ensure a proper durability [1].

The durability of a structure is dependent of the environmental conditions. Those conditions are classified according to Table 4.1 of NP EN 1992-1-1, and depending from the exposure class, the same rule provides a nominal concrete cover on *section 4.4* to ensure a proper durability [5].

This reservoir is in the city of Tavira, Portugal, and does not contain aggressive chemicals. On this way, the 1st, 3rd and 5th exposure classes are not considered and the 2nd, 4th and 6th classes are respectively XC4, XS1 and XA1. The tank should has 45mm of nominal concrete cover.

2.2 Crack control

Cracking is normal in concrete structures, but those cracks should be controlled to ensure a proper operation of the structure. To perform that control, NP EN 1992-1-1 defines that crack width (w_k) should be limited to w_{max}, which values are given in Table 7.1N of the same rule [5].

However, reservoirs have tighter criteria to accomplish, given by EN 1992-3. Reservoirs are classified regarding the tightness according with Table 7.105, and then should be verified the conditions present on article (111*)* [6]. This reservoir is classified as tightness class 1, so the cracks width should be limited to w_{k1}. To the elements in contact with water the cracks width are limited to w_{k1} while the other elements are limited to w_{max}.

The crack widths could be obtain with or without direct calculation. In this work is chosen the first option, following the methodology present in section 7.3.3 of EN 1992-3 [6].

2.3 Materials

Known the exposure class in 2.1 is chosen a concrete C30/37 [5] [9], which properties are in Table 1.

Table 1: Concrete properties

f_{ck} [MPa]	f_{cd} [MPa]	f_{ctm} [MPa]	E_c [GPa]	ε_{c2} [‰]	ε_{cu2} [‰]	α [ºC^{-1}]
30	20	2,9	33	2,0	3,5	10^{-5}

To avoid the direct contact with the soil, it is put a layer below of the bottom slab with poor concrete, with the strength class C12/15.

To the reinforcement, is chosen a steel S500, which properties are in Table 2.

Table 2: Reinforcement properties

f_{yk} [MPa]	f_{yd} [MPa]	E_s [GPa]	ε_{yd} [‰]
500	435	210	2,18

2.4 Actions

The design's actions to this case study are defined following the articles present in NP EN 1991-1-1 [2] and EN 1991-4 [4].

2.4.1 Direct actions

The direct actions that are applied on this reservoir are shown and quantified on Table 3.

Table 3: Direct actions

Self-weight [kN/m^3]	Light concrete [kN/m^3]	Water impulse [kN/m^3]	Live load [kN/m^2]
25,0	14,7	10,0	0,4

2.4.2 Indirect actions

As indirect actions is necessary to take into account the temperature variations, shrinkage and creep.

2.4.2.1 Thermal loads

According with NP EN 1991-1-5, the thermal effect can be divide in four components: ΔT_U, ΔT_{My}, ΔT_{Mz} and ΔT_E, where the last component can be despise.

To get the first component, it is necessary to take into account that this reservoir is located in the Portuguese city of Tavira [3].

About the second and third components, the NP EN 1991-1-5 does not specifies any value to get these. So, is assumed the same values recommended to concrete pipelines, in article 7.5(3) [3].

The thermal actions to consider to the structural design are shown in Table 4.

Table 4: Thermal actions

T_{min} [ºC]	T_{max} [ºC]	T_0 [ºC]	ΔT_U [ºC]	ΔT_M [ºC]
0	40	20	20	15

2.4.2.2 Shrinkage

The shrinkage is the result of the concrete volume decrease, and has four components: hydric, plastic, thermal and chemical. However, the hydric component is the prevailing one, and so the NP EN 1992-1-1 only considers this component, and gives the expression 3.8 to shrinkage extension [5].

Concrete shrinkage depends on the ambient humidity, the dimensions of the element and the composition of the concrete. In Table 5 are shown parameters to obtain shrinkage extension (ε_{cs}).

Table 5: Parameters to obtain shrinkage extension

RH [%]	h_0 [mm]	t_s [days]	f_{cm} [MPa]	α_{ds1}	α_{ds2}
60	200	28	38	4	0,12

Following the procedures presents in section 3.1.4 and annex B of the NP EN 1992-1-1, it is possible to obtain the time evolution of concrete's shrinkage [5], which stabilizes on the maximum value of 0,41‰.

To quantify the shrinkage in design, it is possible to simulate this indirect action as a uniform temperature load ($\Delta T_{u,cs}$), as specified on equation 1.

$$\Delta T_{u,cs} = \frac{\varepsilon_{cs}}{\alpha} \tag{1}$$

Known the shrinkage extension and the linear coefficient of thermal expansion of concrete (α), it is assumed to this action a value of -40ºC uniform temperature load to all structure, except in the bottom slab, where the assumed value is -20ºC because of the earlier cast of this last element.

2.4.2.3 Creep

As shrinkage, the concrete creep is a time dependent effect. To the same stresses applied since the beginning (t_0), a concrete element increase gradually the deflections. Besides the time loading, this effect depends of other factors, such as load period and intensity, humidity, concrete composition and section geometry.

According with the article 3.1.4(2) of NP EN 1992-1-1, if an element is not subjected to a compressive stress greater than 45% of concrete characteristic compressive cylinder strength (f_{ck}), the creep coefficient could have the value provided by Figure 3.1 of the same rule [5].

To obtain the temporal evolution of creep coefficient, the design should follow the procedures presents in section 3.1.4 and annex B of the NP EN 1992-1-1. To perform that, there are shown parameters in Table 6.

Table 6: Parameters to obtain creep coefficient

RH [%]	h_0 [mm]	t_0[days]	f_{cm} [MPa]
60	200	28	38

2.4.2.4 Adjusted elasticity modulus

The indirect actions develop stresses in concrete structures because of constrains to extensions. To evaluate those stresses, it is introduced an adjusted elasticity modulus that considers a decrease of element stiffness [11], given by the equation 2.

$$E_{c,ajus}(t,t_0) = \frac{E_c(t_0)}{1+\chi(t,t_0).\frac{E_c(t_0)}{E_{c,28}}.\varphi(t,t_0)} \tag{2}$$

The adjusted elasticity modulus to consider in structural design are shown in Table 7.

Table 7: Adjusted elasticity modulus

Action	Value [ºC]	$E_{c,ajus}$ [GPa]
Shrinkage	-40	12,5
Uniform temperature	-20	16,5
Differential temperature	-15	16,5

2.5 Combinations of actions

In Table 8 are shown the combinations of actions, according with NP EN 1990 [1], to be consider in this work.

Table 8: Combinations of actions

Elements	Fundamental ULS	Seismic ULS	Serviceability state
Structural walls	$E_d = 1{,}35\ pp + 1{,}5\ I_w$	$E_d = A_E + pp + I_w + sc$	$E_d = pp + I_w + ret + 0{,}5\ (\Delta T_u + \Delta T_d)$
Bottom slab	$E_d = 1{,}35\ pp + 1{,}5\ I_w$	-	$E_d = pp + I_w + 0{,}5\ ret$
Cover slab	$E_d = 1{,}35\ (pp + rcp) + 1{,}5\ sc$	-	$E_d = pp + rcp + \psi_2\ sc + ret + 0{,}5\ (\Delta T_u + \Delta T_d)$

2.6 Foundation ground

This soil is mainly composed with fractured siltstone, bellow of a layer with 1 meter of loamy sand. To take advantage of the main layer, the loamy sand layer is removed.

To consider the soil stiffness in design, it is followed the Winkler's hypothesis, where the soil is simulate as a group of springs with linear elastic behaviour [12]. The stresses applied on the soil are given by equation 3, where K_w is the Winkler modulus (springs stiffness) and w_s are the displacements on the several springs.

$$\sigma_s = K_w w_s \tag{3}$$

It is difficult to determine the Winkler modulus. Taking into account the values given by Bowles [13], it is assumed that the siltstone has a Winkler modulus with a value of 100.000 kN/m^3.

3 Structural analysis

3.1 Structural behaviour

3.1.1 Structural walls

The slenderness of a reservoir, given by the ratio between a characteristic dimension of the base and the maximum height of water, is an important parameter to the structural behaviour. This reservoir is in an intermediate level of slenderness, so it is expected important stresses to both directions.

Besides the bending moments and shear forces, the hydrostatic impulse generates tension on the walls. To equilibrate this action, there are reactions on the joints, which represents tension stresses on the connected walls, as illustrated in Figure 1.

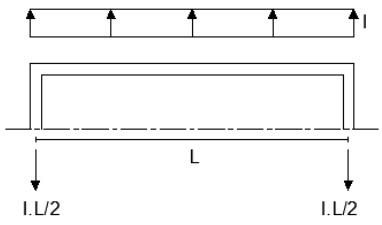

Figure 1: Tension forces in walls

3.1.2 Bottom slab

This element does not have constant thickness. In the areas close to the walls, the bottom slab is thicker while in central area is thinner. In fact, in central area the hydrostatic forces are transmitted to the soil and in this way it is not needed a thick section, while in the other areas the bottom slab has to equilibrate the vertical loads and bending moments from the walls.

3.2 Finite elements models

The analysis was performed using a structural software [18].

All the area elements should be design as 'shell' elements, with square shape and four nodes. The beams and columns should be designed as 'frame' elements.

The walls and cover slab have no restrains. To consider the soil deformability mentioned in chapter 2.6, the bottom slab has springs in all nodes, with a vertical stiffness equal with Winkler modulus.

Regarding the geometry, the bottom slab has different thickness in different areas. The walls have variable thickness from bottom to the top. To consider this variability in design the walls are divided in four walls in their extension, with the dimensions shown in Table 12, where the wall P1 is the closest to the bottom slab and is the thick wall while the wall P4 is the highest and thinnest one.

4 Reservoir analysis

To an easier demonstration of forces, the following figures show the main positions of each elements.

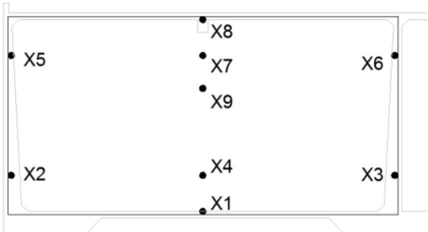

Figure 2: Main sections of Wall X

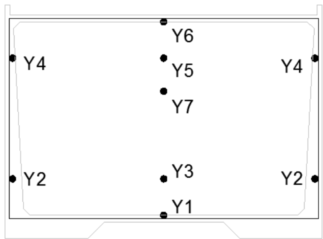

Figure 3: Main sections of Wall Y

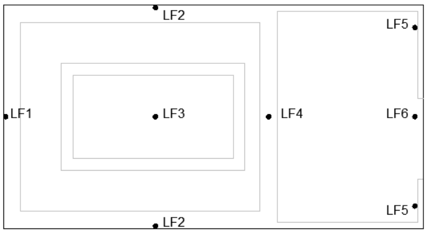

Figure 4: Main sections of bottom slab

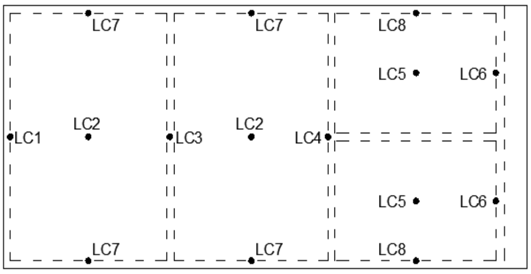

Figure 5: Main sections of cover slab

4.1 Forces

Cracks should be controlled according with the criteria of NP EN 1992-1-1 and EN 1992-3 as announced in chapter 2.2.

In the following Tables are shown the axial forces and bending moments to the main sections of all elements[1], as well the geometry of those sections.

Table 9: Forces in wall X, to horizontal direction

Section	H [m]	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m]	M_{serv} [kNm/m]	N_{serv} [kN/m]
X1	0,00	0,55	0,50	15,8	224,7	-	-24,4	402,5
X2	1,10	0,50	0,45	-216,9	314,3	263,3	-119,5	172,6
X3	1,10	0,50	0,45	-145,0	145,9	186,4	-94,5	184,8
X4	1,10	0,50	0,45	68,0	150,0	-	25,5	217,3
X5	4,75	0,30	0,25	-69,9	125,0	66,3	-36,6	43,1
X6	4,75	0,30	0,25	-50,0	53,0	57,8	-30,3	32,4
X7	4,75	0,30	0,25	51,0	50,0	-	18,8	74,7
X8	5,85	0,25	0,20	7,0	-5,0	-	0,8	15,5

Table 10: Forces in wall X, to vertical direction

Section	H [m]	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
X1	0,00	0,55	0,50	-133,1	-30,0	194,0	-120,0	32,6
X9	3,85	0,35	0,30	100,0	-60,0	-	51,7	0,9

Table 11: Forces in wall Y, to horizontal direction

Section	H [m]	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
Y1	0,00	0,55	0,50	19,7	12,1	-	-14,8	332,6
Y2	1,10	0,50	0,45	-210,3	308,0	256,8	-119,5	149,4
Y3	1,10	0,50	0,45	89,1	170,3	-	38,1	169,8
Y4	4,75	0,30	0,25	-60,8	119,6	70,5	-36,6	48,0
Y5	4,75	0,30	0,25	34,5	145,9	-	19,5	38,5
Y6	5,85	0,25	0,20	9,6	62,9	-	5,5	10,2

Table 12: Forces in wall Y, to vertical direction

Section	H [m]	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
Y1	0	0,55	0,50	-50,0	-190,0	147,8	-86,7	-43,4
Y7	3,85	0,35	0,30	54,3	-157,1	-	36,3	-31,7

Table 13: Forces in bottom slab, to direction X

Section	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
LF1	0,70	0,65	-123,0	193,8	161,8	-118,2	235,6
LF2	0,70	0,65	-30,9	312,4	-	-31,8	267,6
LF3	0,20	0,15	0,8	36,7	-	0,0	85,0
LF4	0,70	0,65	227,1	178,9	253,2	102,6	197,2
LF5	0,95	0,90	-150,0	20,0	-	-33,0	223,6

[1] The positive signal means tractions on the inside fibers to bending moment and tension stresses to axial forces

Table 14: Forces in bottom slab, to direction Y

Section	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
LF1	0,70	0,65	-109,5	59,1	-	-21,5	201,0
LF2	0,70	0,65	-162,4	217,3	148,0	-165,7	161,6
LF3	0,20	0,15	4,0	158,4	-	0,0	117,8
LF4	0,70	0,65	74,8	51,5	-	50,0	125,6
LF6	0,95	0,90	-128,0	-10,0	-	-20,0	315,0

Table 15: Forces in cover slab, to direction X

Section	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
LC1	0,20	0,16	28,0	43,0	34,0	17,0	25,0
LC2	0,20	0,16	-22,0	30,0	-	-13,0	15,0
LC3	0,20	0,16	-11,5	25,0	-	-8,0	7,0
LC4	0,20	0,16	45,0	120,0	120,0	40,0	35,0
LC5	0,20	0,16	-20,0	10,0	-	-9,0	10,0
LC6	0,20	0,16	26,0	30,0	45,9	10,0	45,0

Table 16: Forces in cover slab, to direction Y

Section	t [m]	d [m]	M_{sd} [kNm/m]	N_{sd} [kN/m]	V_{sd} [kN/m	M_{serv} [kNm/m]	N_{serv} [kN/m]
LC2	0,20	0,16	-24,0	21,0	-	-14,0	20,0
LC5	0,20	0,16	-8,0	0,0	-	-3,5	55,0
LC6	0,20	0,16	-6,0	2,5	-	-6,0	85,0
LC7	0,20	0,16	23,0	30,0	35,0	14,0	20,0
LC8	0,20	0,16	18,0	8,0	25,0	10,0	45,0

5 Seismic analysis

5.1 Rules

5.1.1 Parameters to define elastic response spectrum

The NP EN 1998-1 defines an elastic response spectrum to take into account the seismic action in structural design, with a linear single degree of freedom system[7].

For this reservoir the local and respective acceleration are given in Table 17.

Table 17: Reference peak ground acceleration

Seismic action 1		Seismic action 2	
location	a_{gr} [m/s^s]	location	a_{gr} [m/s^s]
1.3	1,5	2.3	1,7

Considering the function of this reservoir the importance class should be IV, and so the NP EN 1998-1 gives an importance factor to obtain the design ground acceleration [7]. The importance factors to this reservoir are shown in Table 18.

Table 18: Importance coefficient

Importance class	Seismic action 1	Seismic action 2
IV	1,95	1,50

The NP EN 1998-1 defines different types of soil, regarding the capacity of propagate the seismic waves. Known that this soil is majority composed with fractured siltstone, the soil can be classified as type A.

At least, the NA of NP EN 1998-1-1 defines other parameters needed to obtain the elastic response spectrum [7], which are shown in Table 19.

Table 19: Parameters to obtain elastic response spectrum

Seismic action	Soil type	S_{max}	T_B [s]	T_C [s]	T_D [s]
1	A	1,0	0,1	0,6	2,0
2		1,0	0,1	0,25	2,0

5.1.2 Behaviour coefficient

A structure has, more or less, capacity to dissipate energy and so don't have an elastic behaviour. To take into account this non-linearity the NP EN 1998-1-1 defines a behaviour coefficient which reduces the displacements and accelerations obtained in elastic response spectrum [7].

A reservoir has a fragile behaviour. According with articles 4.4(1)P and 4.4(3)P of EN 1998-4 the behaviour coefficients to structural and water portions are respectively 1,5 and 1,0 [8].

5.2 Seismic model

When a reservoir is submitted to an earthquake, there is a portion of water that oscillates with the same period of the structure (impulsive portion) and another one (oscillating portion) that oscillates independently from the structure, causing dynamic overpressures.

This work considers the seismic model proposed by Housner [], schematized in Figure 6. The portions of water are simulated as two different masses, which are connected to the structure by a rigid and non-rigid springs respectively to impulsive and oscillating portions.

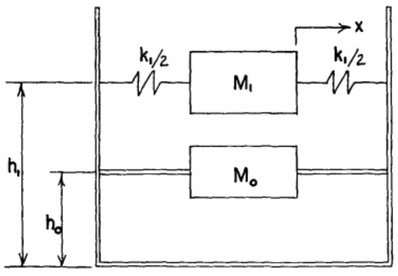

Figure 6: Housner seismic model

5.3 Response spectrum

5.3.1 Structure and impulsive water portion

This non-elevated reservoir has a high stiffness to horizontal displacements, and so the acceleration from response spectrum takes close values to the design ground acceleration. On this way, it is assumed that the spectral acceleration takes the same value as the design ground acceleration. To the two seismic actions, the design spectral accelerations are shown in Table 20.

Table 20: Design spectral accelerations

Seismic action	S_d (T_1) [m/s^2]
1	1,95
2	1,70

5.3.2 Oscillating portion

The oscillating portion vibrates with the fundamental frequency given by expression 4.

$$f_0 = \frac{\omega_0}{2\pi} \quad \textbf{(4)}$$

From the Table 7 of Mendes [12], it is possible to take parameters needed to perform the seismic analysis for oscillating portion. Those parameters are shown in Table 21.

Table 21: Parameters to the seismic analysis

Wall	H/L	M_i/M	H_i/H	M_o/M	H_o/H	w_o^2L/g
X	1,0	0,542	0,375	0,484	0,583	1,453
Y	1,25	0,637	0,375	0,406	0,617	1,522

In the Table 22 are shown the design spectral acceleration and the fundamental frequency and period of this portion.

Table 22: Design spectral acceleration and fundamental frequency and period

Seismic action	Frequency [Hz]	Period [s]	S_d (T_1) [m/s^2]
1	0,249	4,0	0,741
2	0,288	3,5	0,351

5.4 Masses

All the structural mass vibrates when submitted by an earthquake. However there is no need to consider the bottom slab mass because it is pretended obtain the forces in the base of the walls. The masses needed to the analysis, with the respectively highs, are shown in Table 23.

Table 23: Masses

	Direction X		Direction Y	
Portion	Mass [ton/m]	H [m]	Mass [ton/m]	H [m]
Walls	16,1	3,85	12,2	3,55
Cover Slab	10,2		5,1	
Impulsive	36,5	2,20	33,4	2,20
Oscillating	32,6	3,40	21,3	3,60

To find at which high are the structural mass it is done an average according with the expression 5.

$$H = \frac{\sum H_i . M_i}{\sum M_i} \quad \textbf{(5)}$$

5.5 Seismic dynamic analysis through finite elements models

5.5.1 Model

The finite elements model does not allow to define 'spring' elements to connect the water masses to the structure, and so that connection is provided by a 'frame' element without weight but with the stiffness announced in chapter 5.2. However, those elements allow tension stresses, what is physically impossible. On this way, the model considers the impulsive and oscillating masses on one wall to each direction (X and Y). These are 'joint masses' to be inserted in all nodes of the 'shell' elements of the walls, and so the global masses should be equally divided by the existing nodes.

5.5.2 Modal analysis

According with article 4.3.3.3.1(2)P of NP EN 1998-1 all the significant vibration modes should be consider in design [7]. However, will be shown the tree main modes to the situations of a full and empty reservoir.

Table 24: Modal analysis to a full reservoir

			Mass participating factors			
Mode	Period [s]	Frequency [Hz]	UX	UY	SumUX	SumUY
1	0,176	5,67	0,72204	0,00620	0,72204	0,00620
2	0,162	6,16	0,01569	0,52476	0,73774	0,53096
3	0,110	9,10	0,04646	0,06184	0,78420	0,59280

Table 25: Modal analysis to an empty reservoir

			Mass participating factors			
Mode	Period [s]	Frequency [Hz]	UX	UY	SumUX	SumUY
1	0,110	9,10	0,02991	6,77E-07	0,02991	6,77E-07
2	0,108	9,30	5,66E-05	0,47075	0,02996	0,47075
3	0,089	11,26	0,65236	5,37E-05	0,68232	0,47081

From the two Tables above, it is possible to note the importance of the water. In fact the Mode 1 and 3 switch according with the water level in reservoir.

5.6 Seismic forces

The equivalent static force of seismic action results of the multiplication between the design spectral accelerations and masses. The forces of one wall, to each direction, are shown in the Table 26.

Table 26: Forces of one wall

	F_E [kN/m]	
Portion	Direction X	Direction Y
Structure	25,7	98,9
Impulsive	35,6	78,3
Oscillating	24,2	82,3

To input these forces into finite elements model, is necessary to divide them equally as 'area forces', taking into account how many 'shell' elements a wall has. The shear forces and bending moments of seismic combination to ULS are shown in Table 27.

Table 27:Forces to seismic combination

Section	V_{sd} [kNm/m]	M_{sd} [kN/m]
X1	181,0	-130,7
X9	10,0	83,9
Y1	151,8	-100,4
Y7	6,5	64,8

The forces to this combination are equal or lower than the obtained to fundamental combination. This way it is despised this combination to verify the ULS security.

6 Structural design

6.1 Serviceability State

To the cracking control mentioned in chapter 2.3. Known the forces and the geometry from each section it is possible obtain, iteratively, the reinforcement needed. The following Tables show the cracking control to all main sections, where A_{s1} is the reinforcement of internal surface and A_{s2} of the opposite surface.

Table 28: Cracking control in wall X, to horizontal direction

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	h_o/t	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
X1	-24,4	402,5	ϕ 16 // 0,10	ϕ 16 // 0,10	0,40	16	10,6	127,1	0,0	0,17	0,16
X2	-119,5	172,6	ϕ 16 // 0,20 + ϕ 20 // 0,20	ϕ 16 // 0,20	0,57	20	11,7	145,0	-4,9	0,17	0,14
X3	-94,5	184,8	ϕ 16 // 0,20 + ϕ 20 // 0,20	ϕ 16 // 0,20	0,57	20	11,7	124,2	-3,7	0,17	0,12
X4	25,5	217,3	ϕ 16 // 0,20	ϕ 16 // 0,10	0,45	16	11,7	85,7	0,0	0,17	0,11
X5	-36,6	43,1	ϕ 16 // 0,10	ϕ 16 // 0,20	0,80	16	19,5	91,4	-4,6	0,13	0,07
X6	-30,3	32,4	ϕ 16 // 0,10	ϕ 16 // 0,20	0,80	16	19,5	74,8	-3,8	0,13	0,06
X7	18,8	74,7	ϕ 16 // 0,20	ϕ 16 // 0,10	0,80	16	19,5	61,0	-2,2	0,13	0,05
X8	0,8	15,5	ϕ 12 // 0,10	ϕ 12 // 0,10	0,57	12	23,4	11,5	-0,1	0,11	0,01

Table 29: Cracking control in wall X, to vertical direction

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	h_o/t	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
X1	-120,0	32,6	ϕ 16 // 0,10	ϕ 16 // 0,20	0,40	16	10,6	135,8	-5,0	0,17	0,13
X9	51,7	0,9	ϕ 16 // 0,20	ϕ 16 // 0,10	0,67	16	16,7	93,4	-4,9	0,14	0,08

Table 30: Cracking control in wall Y, to horizontal direction

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	h_o/t	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
Y1	-14,8	332,6	ϕ 16 // 0,10	ϕ 16 // 0,10	0,40	20	10,6	99,1	0,0	0,17	0,13
Y2	-119,5	149,4	ϕ 16 // 0,20 + ϕ 20 // 0,20	ϕ 16 // 0,20	0,57	20	11,7	140,3	-4,8	0,17	0,13
Y3	38,1	169,8	ϕ 16 // 0,20	ϕ 16 // 0,10	0,44	16	11,7	86,9	0,8	0,17	0,08
Y4	-36,6	48,0	ϕ 16 // 0,10	ϕ 16 // 0,20	0,80	16	19,5	92,7	-4,5	0,13	0,07
Y5	19,5	38,5	ϕ 16 // 0,20	ϕ 16 // 0,20	0,39	16	19,5	99,4	-3,1	0,13	0,12
Y6	5,5	10,2	ϕ 12 // 0,10	ϕ 12 // 0,10	0,57	16	23,4	34,9	-1,4	0,11	0,04

Table 31: Cracking control in wall Y, to vertical direction

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	h_o/t	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
Y1	-86,7	-43,4	ϕ 16 // 0,10	ϕ 16 // 0,20	0,40	16	10,6	81,9	-3,7	0,17	0,08
Y7	36,3	-31,7	ϕ 16 // 0,20	ϕ 16 // 0,10	0,65	16	16,7	56,0	-3,3	0,14	0,05

Table 32: Cracking control in bottom slab, to direction X

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	h_o/t	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
LF1	-118,2	235,6	ϕ 16 // 0,10	ϕ 16 // 0,20	0,31	16	8,4	154,7	-2,5	0,18	0,15
LF2	-31,8	267,6	ϕ 16 // 0,10	ϕ 16 // 0,20	0,31	16	8,4	92,9	0,0	0,18	0,19
LF3	0,0	85,0	ϕ 12 // 0,10	ϕ 12 // 0,10	0,75	12	29,3	37,6	0,0	0,08	0,06
LF4	102,6	197,2	ϕ 16 // 0,10	ϕ 16 // 0,10	0,31	16	8,4	132,5	-2,2	0,18	0,13
LF5	-33,0	223,6	ϕ 12 // 0,10	ϕ 12 // 0,10	0,13	12	6,2	110,9	0,0	0,30	0,19

Table 33: Cracking control in bottom slab, to direction Y

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	h_o/t	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
LF1	-21,5	201,0	ϕ 16 // 0,10	ϕ 16 // 0,20	0,31	16	8,4	67,8	0,0	0,18	0,16
LF2	-165,7	161,6	ϕ 16 // 0,10	ϕ 16 // 0,20	0,31	16	8,4	174,3	-4,3	0,18	0,17
LF3	0,0	117,8	ϕ 12 // 0,10	ϕ 12 // 0,10	0,75	12	29,3	52,1	0,0	0,08	0,08
LF4	50,0	125,6	ϕ 16 // 0,10	ϕ 16 // 0,10	0,31	16	8,4	72,1	0,9	0,18	0,07
LF6	-20,0	315,0	ϕ 12 // 0,10	ϕ 12 // 0,10	0,13	12	6,2	160,1	0,0	0,30	0,26

Table 34: Cracking control in cover slab, to direction X

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
LC1	17,0	25,0	ϕ 12 // 0,20	ϕ 12 // 0,20 + ϕ 16 // 0,20	1,05	12	89,6	-5,6	0,30	0,06
LC2	-13,0	15,0	ϕ 12 // 0,20	ϕ 12 // 0,20	0,35	12	162,4	-6,1	0,30	0,17
LC3	-8,0	7,0	ϕ 12 // 0,20	ϕ 12 // 0,20	0,35	12	97,8	-3,8	0,30	0,10
LC4	40,0	35,0	ϕ 12 // 0,20	ϕ 12 // 0,20 + ϕ 16 // 0,20	1,05	12	201,9	-13,2	0,30	0,17
LC5	-9,0	10,0	ϕ 12 // 0,20	ϕ 12 // 0,20	0,35	12	112,0	-4,2	0,30	0,12
LC6	10,0	45,0	ϕ 12 // 0,20	ϕ 12 // 0,20	0,38	12	171,2	-5,4	0,30	0,19

Table 35: Cracking control in cover slab, to direction Y

Section	M_{serv} [kNm/m]	N_{serv} [kN/m]	A_{s1}	A_{s2}	ρ [%]	ϕ [mm]	σ_s [MPa]	σ_c [MPa]	$w_{k,lim}$ [mm]	w_k [mm]
LC1	-14,0	20,0	ϕ 12 // 0,10	ϕ 12 // 0,20	0,71	12	93,3	-4,8	0,30	0,07
LC2	-3,5	55,0	ϕ 12 // 0,20	ϕ 12 // 0,20	0,35	12	92,9	-1,5	0,30	0,10
LC3	-6,0	85,0	ϕ 12 // 0,10	ϕ 12 // 0,20	0,38	12	164,5	-3,2	0,30	0,18
LC4	14,0	20,0	ϕ 12 // 0,20	ϕ 12 // 0,20 + ϕ 16 // 0,20	1,05	12	73,5	-4,6	0,30	0,05
LC5	10,0	45,0	ϕ 12 // 0,20	ϕ 12 // 0,20	0,38	12	171,8	-6,2	0,30	0,19
LC6	-14,0	20,0	ϕ 12 // 0,10	ϕ 12 // 0,20	0,71	12	93,3	-4,8	0,30	0,07

6.2 Ultimate Limit State

6.2.1 Bending with axial force

The NP EN 1992-1-1 defines hypothesis to verify the ultimate resistance to bending with axial force [5]. As in the chapter 6.1 this work use a macro in Excel to perform the security verification to ULS, according with the reinforcement choose in previous chapter. This verification is performed to the wall X, as shown in Tables 36 and 37.

Table 36: ULS resistance in wall X, to horizontal direction

Section	M_{sd} [kNm/m]	N_{sd} [kN/m]	μ	$M_{rd,max}$ [kNm/m]	$M_{rd,min}$ [kNm/m]
X1	15,8	224,7	0,004	364,5	-364,5
X2	-216,9	314,3	0,054	129,8	-408,4
X3	-145,0	145,9	0,036	165,2	-442,9
X4	68,0	150,0	0,017	342,2	-163,6
X5	-69,9	125,0	0,066	96,6	-185,7
X6	-50,0	53,0	0,040	104,1	-193,2
X7	51,0	50,0	0,041	193,5	-104,4
X8	7,0	-5,0	0,009	96,0	-96,0

Table 37: ULS resistance in wall X, to vertical direction

Section	M_{sd} [kNm/m]	N_{sd} [kN/m]	μ	$M_{rd,max}$ [kNm/m]	$M_{rd,min}$ [kNm/m]
X1	-133,1	-30,0	0,027	223,4	-423,7
X9	100,0	-60,0	0,056	229,5	-118,6

From the reading of the previous Tables, it is possible to note that this verification is clearly secured. The other sections of all elements verify as well the security to ULS.

6.2.2 Shear force

To verify the shear resistance, this work follow the Portuguese regulation REBAP [10].

Table 38: Shear resistance in wall X, to horizontal direction

Section	V_{sd} [kN/m]	$(A_s/s)_{,min}$ [cm^2/m]	$(A_s/s)_{,adop}$	V_{cd} [kN/m]	V_{wd} [kN/m]	V_{Rd} [kN/m]	$\tau_2.b_w.d$ [kN/m]	V_{Rd}/V_{sd}
X2	263,3	8,0	ϕ8 // 0,125	375,0	157,4	532,4	2500,0	2,02
X3	186,4	8,0	ϕ8 // 0,125	337,5	141,6	479,1	2250,0	2,57
X5	66,3	8,0	ϕ8 // 0,125	187,5	78,7	266,2	1250,0	4,01
X6	57,8	8,0	ϕ8 // 0,125	187,5	78,7	266,2	1250,0	4,61

Table 39: Shear resistance in wall X, to vertical direction

Section	V_{sd} [kN/m]	$(A_s/s)_{,min}$ [cm^2/m]	$(A_s/s)_{,adop}$	V_{cd} [kN/m]	V_{wd} [kN/m]	V_{Rd} [kN/m]	$\tau_2.b_w.d$ [kN/m]	V_{Rd}/V_{sd}
X1	194,0	8,0	ϕ8 // 0,125	382,7	157,4	540,1	2500,0	2,78
X8	57,2	8,0	ϕ8 // 0,125	232,9	94,4	327,3	1500,0	5,72

As in chapter 6.2.1 the resistance to shear forces is clearly verified to all main sections of this reservoir.

7 Conclusions

As verified in this work the reservoir structural design was clearly conditioned by cracking control. These structures have to assure with more or less importance tightness, and so the EN 1992-3 defined rigorous criteria [6].

With tight criteria to the cracking control, the ULS security was clearly verified to the bending moments. The shear resistance was also clearly verified.

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