Structural and Non-Structural Assessment of a RC Water Tank

João Veludo
CERIS; Polytechnic Institute of Leiria/ Department of Civil Engineering, Leiria, Portugal
Email: joao.veludo@ipleiria.pt

Paulo A. L. Fernandes *[+], Hugo Rodrigues**[+], Florindo Gaspar***[+], Nuno Almeida[+], João Baptista*[+], Nuno Martinho[****++]
* CERIS; [+]Polytechnic Institute of Leiria/ Department of Civil Engineering, Leiria, Portugal
** RISCO; [+]Polytechnic Institute of Leiria/ Department of Civil Engineering, Leiria, Portugal
*** CDRSP; [+]Polytechnic Institute of Leiria/ Department of Civil Engineering, Leiria, Portugal
**** [++]Be Water S.A./ Department of technical Servives, Ourém, Portugal
Email: paulo.fernandes@ipleiria.pt; hugo.f.rodrigues@ipleiria.pt; florindo.gaspar@ipleiria.pt; nuno.f.almeida@ipleiria.pt; joao.baptista@ipleiria.pt; nuno.martinho@bewater.com.pt

***Abstract*—This work presents a case study that aimed to assess the structural safety of a RC water tank located in Fátima, Ourém Municipality – Portugal. To perform the assessment an initial survey was conducted and supported by a non-destructive test campaign that allowed the development of a numerical simulation that will allow to evaluate the safety level and to support the definition of the structural retrofitting and rehabilitation needs.**

***Index Terms*— water tank, structural retrofitting, *in-situ* testing, finite element analysis**

I. Introduction

In Portugal, it has been a common practice in the construction of tanks to use regular or prestressed reinforced concrete due to its lower costs when compared with steel structures.

In this type of structures, it is necessary to fulfill certain requirements such as the control of cracking, which is directly related to its durability and to prevent the percolation of the liquid contained through the structural elements or infiltration of rainwater into the tank.

Given this, due to the pathologies that this reservoir presented, it was clear it needed to be inspected and retrofitted. The seismic load, not considered in the initial design stage, was analyzed, once the seismic performance of the water tank is important and dependent on the earthquake characteristics and frequency of water tank. The maximum response in terms of stresses, displacement and hydrodynamic pressure was analysed.

II. Visual Inspection and Data Collection

The water tank under study was built in 1967 by the religious order "Monfortinos". At the present time it is managed by the water concessionary Be Water S.A. - Águas de Ourém. It has a rectangular and axisymmetric geometry divided into two cells. Each cell has 34 m long, 19 m wide and has a height of 4,1 m between the bottom slab and the top slab, a partially precast rib and block slab, as shown in Figure 1.

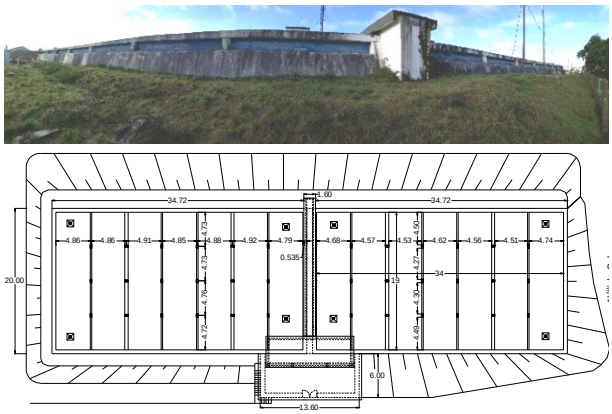


Figure 1. Water tank

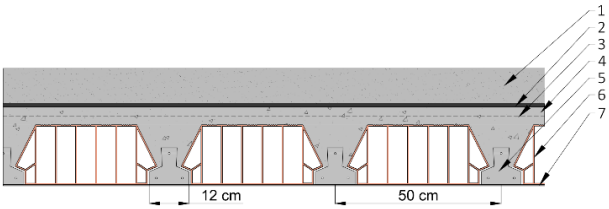
The structure is made of six (6) transversal reinforced concrete frames, composed by columns, oriented in two orthogonal directions, rectangular beams, lightweight slab and outer walls. These walls have a trapezoidal shape in cyclopean concrete, with a variable width varying from 1.80 m at the bottom and 1,0 m at the top 1,0 m) and are 4,1 m tall. The one-way slab's precast beams are 50 cm apart from each other and are supported by the concrete frame. Figure 2 illustrates the cross-section of the slab and TABLE I lists the geometry of all structural elements.

In 2008 the exterior slab was waterproofed by an asphalt membrane covered with a mortar layer, and the

Manuscript received April 5, 2018; revised August 1, 2018.

doi: 10.18178/ijscer.7.3.204-212

existing damages in the structure were also repaired. The present work was requested by Be Water S.A. in 2017 due to the recently pathologies detected, with the main purpose to identify and establish corrective measures if needed.



1 – Mortar*; 2 – Asphalt membrane*; 3 – Welded steel wire mesh; 4 – Concrete; 5 – Prestressed precast beams; 6 – Clay block; 7 – Cement plaster
*Post-2008 intervention

Figure 2. Top slab cross-section

TABLE I. STRUCTURAL ELEMENTS DIMENSIONS

Element	ID	Material	Width [m]	Height [m]	Spacing [m]	Thickness [m]
Top slab						
Precast beams	-	Concrete	0,12	0,12	0,50	-
Slab	L1/L2		-	-	-	0,20
Concrete frame						
Columns	P1	Concrete	0,20	0,45	-	-
	P2		0,45	0,20	-	-
Beams	V1		0,20	0,50	-	-
	V2		0,45	0,50	-	-
Outer walls	Par1	Cyclopean concrete	Top: 1,00 Bottom: 1,80	4,10	-	-

A. *Damage Assessment*

The visual inspection of the tank allowed an overall assessment of its conservation status. No important damages were detected in the structural elements inspected, and it may even be stated that the tank is, generally, in good structural condition. The main identified pathology is related with the cracking of the top slab that allows the infiltration of rainwater into the tank. Some non-structural damages were also observed, which are described next.

Outside the tank it was possible to observe high levels of cracking on the exterior of all walls, which is more pronounced in the non-structural vertical elements, as showed in Figure 3, and in the interface between the walls and the slab. It is also important to note the presence of vegetation in some points of the tank next to the waterproofing membrane of the technical gallery (Figure 4), at the top of the same gallery and next to the roof of the entrance of the tank.

In the top slab was observed cracking with a regular pattern (Figure 5). This pathology has an impact on the tank due to the water infiltrations.

Figure 3. Cracking in a non-structural column

Figure 4. Vegetation

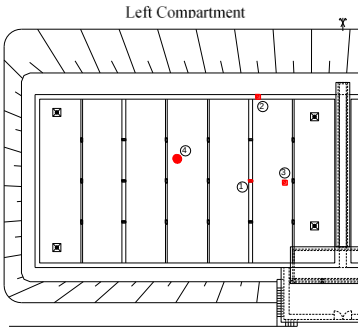
Figure 5. Cracking of the top slab - Overview

In the interior, there are detachments of the wall's plaster, cracking, and infiltrations caused by the damages in the slab over the entire length of the structure (Figure 6).

Figure 6. Pathology: Cracking/Infiltration

B. *In-situ Tests*

To support the visual survey performed and to improve the knowledge about the structure behavior a campaign of non-destructive tests was planned and performed in the structure with the layout shown in Figure 7.



1 (Column): Rebar detection; Rebound test; Windsor probe
2 (Wall): Rebar detection; Core drilling; Rebound test; Windsor probe
3 (Bottom slab): Rebar detection; Windsor probe
4 (Top Slab): Core drilling

Figure 7. *In*-situ tests layout

1) *Material characterization*

For the mechanical characterization of the structural materials non-destructive and semi-destructive tests were carried out: (i) surface hardness test to evaluate the rebound number and the homogeneity of the concrete; (ii) penetration resistance test (Windsor Probe) to evaluate the compressive strength of concrete; and (iii) extraction of concrete core samples to evaluate compressive strength, definition of the slab height, identification of its constituent elements, and to verify the state of conservation of the materials.

The test rebound number test was performed following the requirements of the standard EN 12504-2 [1].

From these tests, the nominal values obtained for the compressive strength were of 26,0 MPa for the concrete frames and 40,0 MPa for the cyclopean concrete walls.

The penetration resistance tests were performed with the Windsor pistol, model HP Windsor Probe System, according to the American standard ASTM 803-3 (2010) [2].

From these tests, the nominal values obtained for the compressive strength were of 26,0 MPa for the concrete frame and 32,0 MPa for the cyclopean concrete walls.

The concrete core samples extracted from the structure, as seen in Figure 8, were prepared and tested according to the standard EN 12504-1:2009 [3]. From this test, the compressive strength obtained was of 37,7 MPa.

Figure 8. Core drilled concrete sample

2) *Ambient vibration testing and dynamic identification*

The purpose of the dynamic characterization was to evaluate the dynamic characteristics of the structure, i.e. the frequencies and respective vibration modes, through an environmental vibration test.

To verify the accuracy of the numerical models in representing the structural behavior is the application of vibrational methods to identify the structural dynamic properties. These dynamic properties, such as modal natural frequencies, modal shapes and damping of a structure, can provide a direct correlation between the physical properties and the numerical modes [4, 5].

For this purpose, measurements of the structure were made, without forced vibration, aiming to obtain the modal response to support the development and calibration of a numerical model according to the layout shown in Fig. 9. The measured frequencies are listed in TABLE and the corresponded modes of vibration are represented in Fig. 10.

TABLE II. NATURAL FREQUENCIES

Frequency [Hz]	Vibration Mode Type	Damping [%]
27.1	Transversal/Torsional	0.85
31.0	Global Transversal	1.34
46.7	Transversal/Torsional	0.81

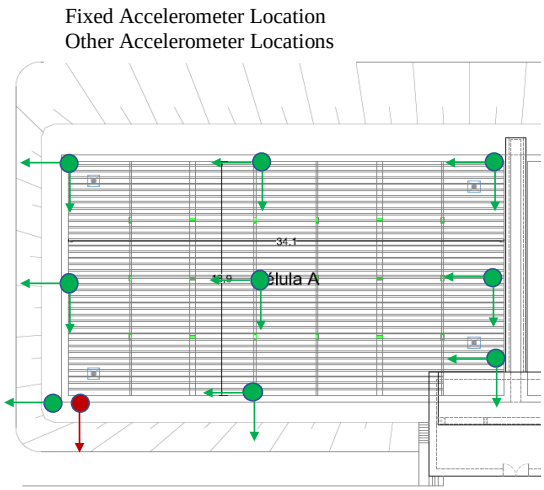
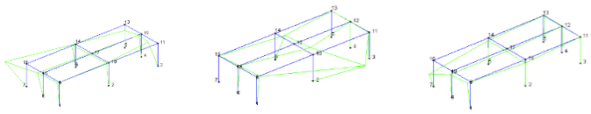


Figure 9. Accelerometer locations and measurement direction



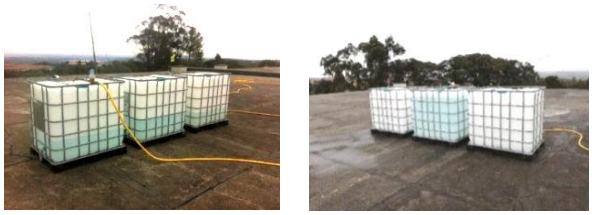
a) Mode 1 (27,1 Hz) b) Mode 2 (31,0 Hz) c) Mode 3 (46,7 Hz)

Figure 10. Natural vibration shape modes representation

3) *Static load test*

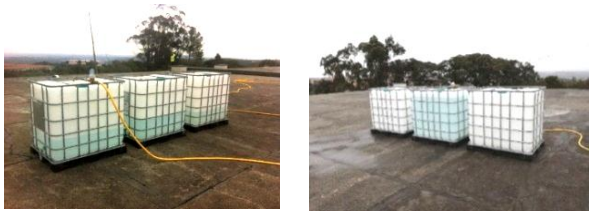
The static load test was performed to evaluate the structural behavior of the top slab for different load levels. It was intended to determine the deformation imposed by each load level and to obtain an estimate of the flexural stiffness to calibrate the numerical model.

For this purpose, 3 plastic tanks were used, with a capacity of approximately 1000 liters each and dimensions of 1,20 x1,00 m^2. They were placed on a 1 m strip on the alignment of two columns. Thirteen load levels were considered between 180 l (60 l in each tank) and 1063 l (only in the central tank), divided into two phases. These load levels are represented in Table and Fig. 11.



a) Phase I b) Phase II

Figure 11. Load water distribution in the tanks



c) Phase I d) Phase II

Figure 11. Load water distribution in the tanks

TABLE III. LOAD LEVELS DISTRIBUTION (IN LITERS)

Phase	Load Level	Tank 1	Tank 2	Tank 3	Total
I	1	60	60	60	180
	2	120	120	120	360
	3	180	180	180	540
	4	240	240	240	720
	5	300	300	300	900
	6	360	360	360	1080
II	7	0	360	0	360
	8	0	480	0	480
	9	0	600	0	600
	10	0	720	0	720
	11	0	840	0	840
	12	0	960	0	960
	13	0	1063	0	1063

All deformations were recorded and the results are presented in Fig. 12 and Fig.13 for each loading phase. These results allowed to estimate the bending stiffness of the slab (37600 kN.m^2/m) and verify that it is above the value presented in the manufacturer's catalog (30388 kN.m ²/m).

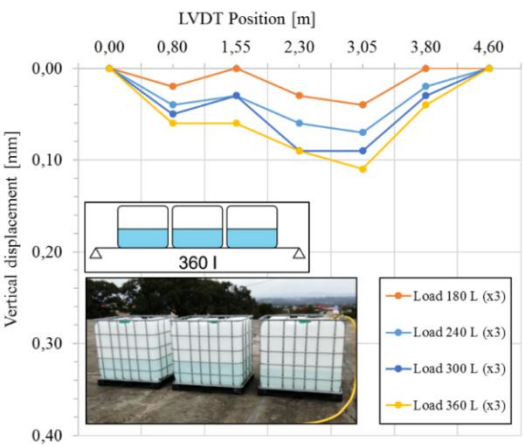


Figure 12. Phase I Load/Displacement diagram

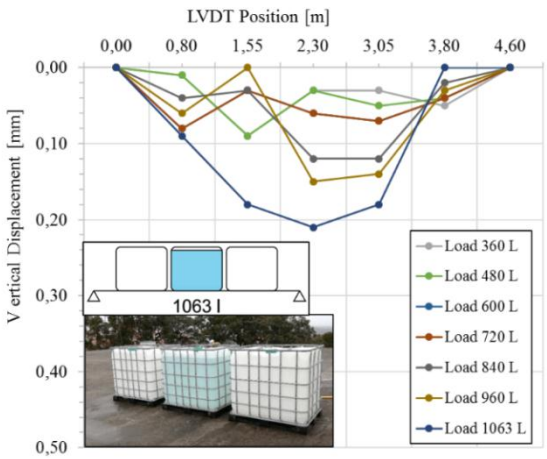


Figure 13. Phase II Load/Displacement diagram

III. NUMERICAL ANALYSIS

A. Material Definition

Based on the data collection performed throughout the *in-situ* and laboratory tests, a C25/30 concrete class was assumed for the structural verifications, with a mean value of concrete cylinder compressive strength of 33 MPa. Although there was no clear test performed that could give a clear knowledge of the type of steel used, mainly on the beams, it was assumed an A24 class of plain steel with a tensile strength of 240 MPa, that represents the current practice back in 1967 [6, 7].

B. Loads

The loads considered for the numerical model are presented in TABLE II and Table V.

TABLE II. DEAD LOADS CONSIDERED - MATERIALS

Material	kN/m ³
Steel	77.0
Water	10.0
Mortar	20.0
Reinforced Concrete	25.0
Cyclopean Concrete	24.0
Soil	16.0

TABLE III. DEAD LOADS CONSIDERED – OTHER LOADS

Description	kN/m ²
Top Slab:	
Mortar	2.00
Asphalt membrane	0.04
Concrete	1.50
Clay blocks	0.75
Prestressed precast beams	0.40
Cement plaster	0.06
Outer walls:	
Waterproof coating	0.06

The live loads that were considered are presented in TABLE IV.

TABLE IV. LIVE LOADS CONSIDERED

Description	Value
Rooftop imposed load	1.00 kN/m ²
Temperature (ΔTu)	±10 ºC
Earthquake	

Both types of earthquake were considered (moderate and high magnitude, according with Portuguese national standard [7]-[9]), a rock or other rock-like geological formation was assumed for the ground type. The response spectrum analysis was carried based on a response spectrum with a behavior factor of 1,5 to account for the non-linear response of the structure. Also, damping coefficients of 5% and 0,5% were used for the concrete structure and the water mass, respectively [10, 11].

The use of a different damping coefficient for the water mass was needed to calculate the influence of the seismic action on the water mass and consequently the sloshing effect of the water on the exterior walls and possibly on the top slab. The *Housner* method [12,13] was used to quantify the dynamic action that the earthquake causes on the water and tank as well as the height of the sloshing wave ($d_{máx}$). In Figure 14 are represented the geometrical properties for the tank alongside with the representation scheme that assumes that the lower mass of water (M_i) is attached rigidly to the tank at a proper height (h_i), and a secondary mass of water (M_o) will be excited by the ground (and tank) motion causing it to oscillate. This will exert an oscillating force on the tank's walls, represented by the pressure diagram (p_i) and equivalent springs. These dynamic forces were added to the static forces.

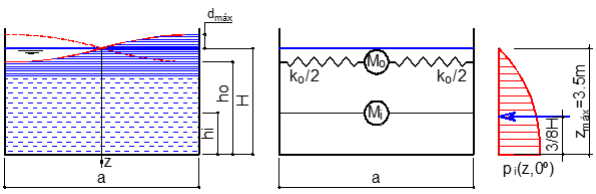


Figure 14. Representation of the Hydrodynamic behavior

C. Finite Element Model

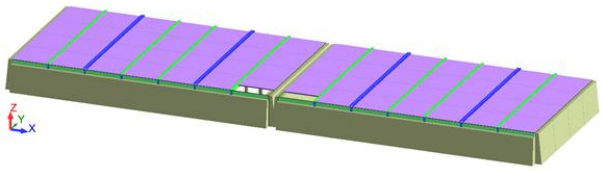
A 3D numerical model was developed, composed of bar elements, for the beams and columns, and 4 node plane quadrilateral finite elements for the slabs and walls, as presented in Fig. 15 a) to c). All supports in the base were considered with the displacement fixed.

The frequency of the three horizontal shape modes achieved with this model are presented in Table V.

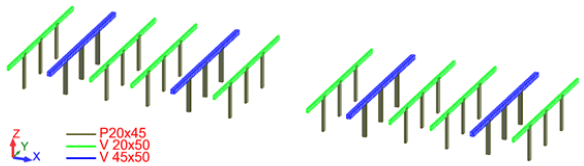
TABLE V. FREQUENCY, PERIOD AND EFFECTIVE MODAL MASS PARTICIPATION

Shape Mode	Frequency [Hz]	Period [s]	Modal mass participation [%]		
			X-X	Y-Y	Z-Z
1	27.12	0.0369	0.01	23.58	0
2	27.29	0.0366	0.08	23.75	0
3	28.36	0.0353	4.87	23.75	0

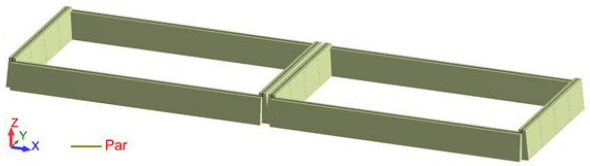
Vertical frequencies were not considered for the slab because they were higher than 10 Hz, which may be disregarded according to national standards [8]. Since these results are close to those obtained with the *in-situ* tests (see TABLE) it was concluded that the numerical model developed was adequate for the remaining analysis, mainly the structural verification.



a) Global model



b) Concrete frames



c) Exterior walls

Figure 15. Numerical model

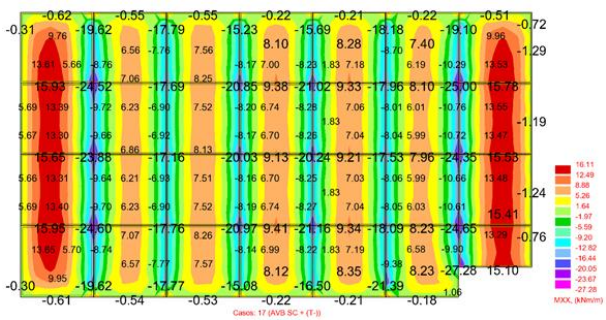
IV. RESULTS

Considering all the loads described and the loads combination prescribed in the national standards for Ultimate Limit State (ULS) and for Serviceability Limit State (SLS) the structure was analyzed and the results are presented and discussed next.

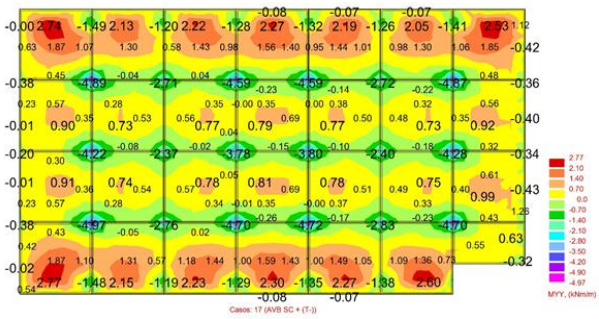
A. Ultimate Limit State (ULS)

1) Top Slab

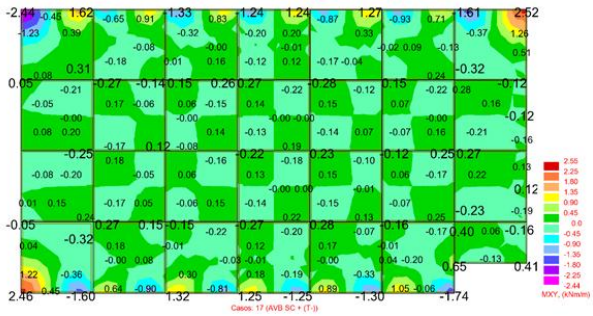
The maps for the resulting bending forces on the top slab, for the most demanding combinations, are presented in, Fig. 16 and for the shear forces in Fig. 17.



a) Bending Moment MXX



b) Bending Moment MYY



c) Bending Moment MXY

Figure 16. Bending Forces

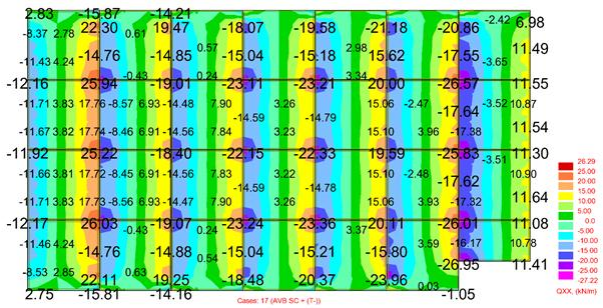


Figure 17. Shear Forces XX Direction

The comparative results between the acting forces and the resisting forces are presented in TABLE VI. The sum of the bending moment (M_{XX}) with the torsional moment (M_{XY}) was considered for the total acting bending moment.

The principal tensile stresses acting on the top slab are presented in Figure 18.

TABLE VI. ULS VERIFICATION FOR THE TOP SLAB

Acting design force		Resisting force
$M_{d,XX}$ [kN.m/m]	15.93	$M_{Rd,XX}$ [kN.m/m]
$M_{d,XY}$ [kN.m/m]	0.46	
$M_{d,Total}$ [kN.m/m]	16.39	51,30
		V_{Rd} [kN/m]
V_d [kN/m]	28.60	42.50

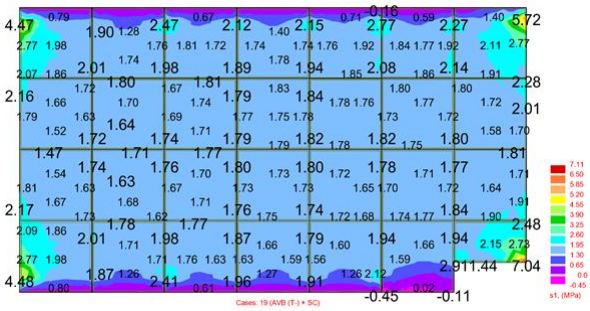
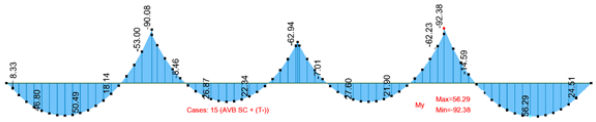


Figure 18. Principal tensile stresses acting on the top slab

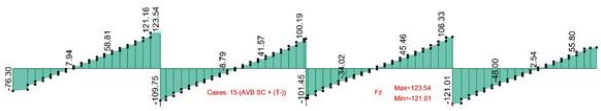
According to the limit states analysis the top slab was considered safe.

2) Beams and Columns

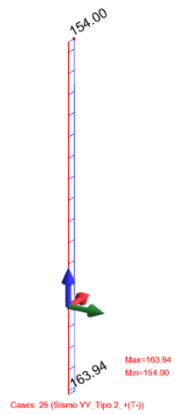
When performing the visual inspection and damage assessment described earlier, no structural or non-structural damage were detected on the beams and columns, therefore the design stress diagrams computed with the numerical model are presented in Figure 19.



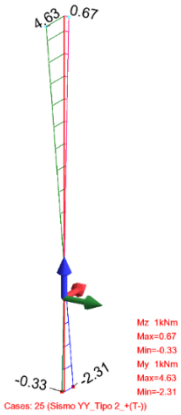
a) Bending moment: 0,20 x 0,50 m ²Beam [kN.m]



b) Shear force: 0,20 x 0,50 m ²Beam [kN]



c) Axial force: 0,20 x 0,45 m ²Column [kN]



d) Bending moment: 0,20 x 0,45 m ²Column [kN.m]

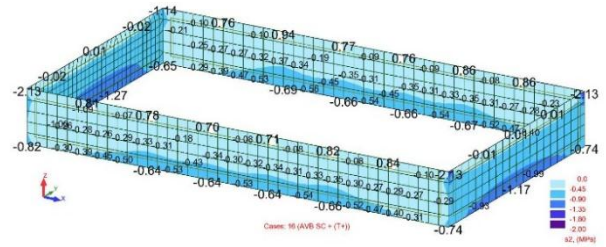
Figure 19. Maximum forces acting on the beams and columns

The ULS design stress on the beams were medium to low, justifying the inexistence of pathologies. The safety factor for the columns in bending with axial force were proper to justify the good safety conditions of the columns.

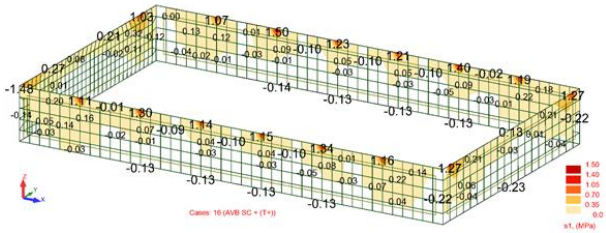
3) Exterior walls

The verification of the cyclopean concrete walls was performed by comparing the compressive and tensile stresses on the walls due to adequate ULS load

combinations with the corresponding capacity of the concrete. The principal stresses on the walls are presented in Fig.20.



a) Principal compression stresses



b) Principal tension stresses

Figure 20. Principal stresses on the exterior walls due to ULS load combinations

The maximum acting compressive stress of 1,50 MPa and the maximum acting tensile stress of 1,35 MPa proved to be much lower than the capacity of the concrete (16,7 MPa for compression and 2,20 MPa for tension), consequently verifying its safety.

B. *Serviceability Limit State (SLS)*

1) Crack control

When performing the visual inspection, the main pathology detected was the cracking of the top slab. The bending moments for the frequent load combination are presented in Fig. 21.

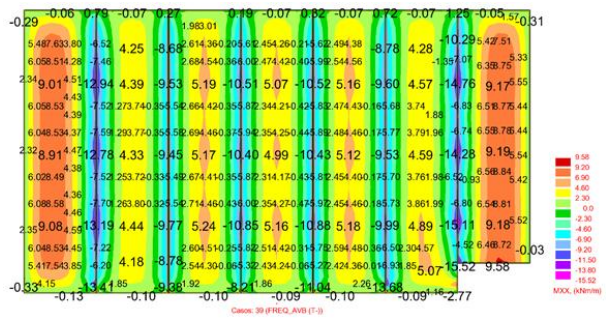


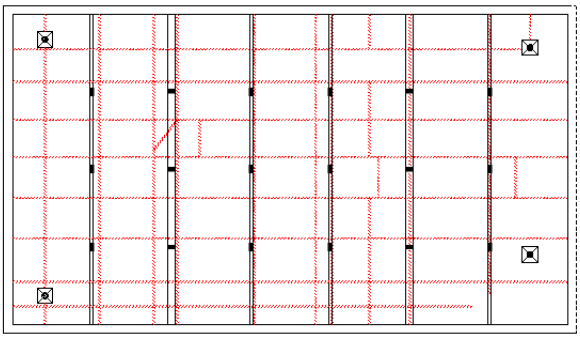
Figure 21. Bending moments for the frequent load combination [kN.m/m]

In Table VII are presented both positive and negative bending moments for the frequent load combination, the corresponding capacity of the one-way rib and block slab and the negative cracking moment.

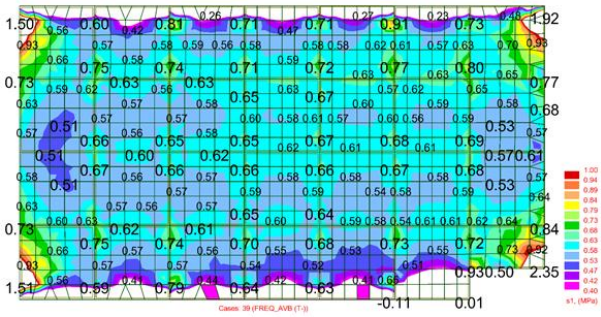
TABLE VII. CRACKING VERIFICATION

Positive Bending Moment		Negative Bending Moment	
M_{freq} [kN.m/m]	M_{fctk} [kN.m/m]	M_{freq} [kN.m/m]	M_{cr} [kN.m/m]
9,58	35,1	-15,11	-5,63

The cracking verification is verified for the positive bending moments, but for the negative moments it is not. These results explain most of the cracking pattern observed on the top of the slab, and above the frames. Fig. 22 shows the cracking pattern observed on the top slab as well as the principal tensile stresses due to the temperature action.



a) Exterior cracking pattern observed on the top slab



b) Principal tensile stresses on the top slab

Figure 22. Cracking pattern of the top slab

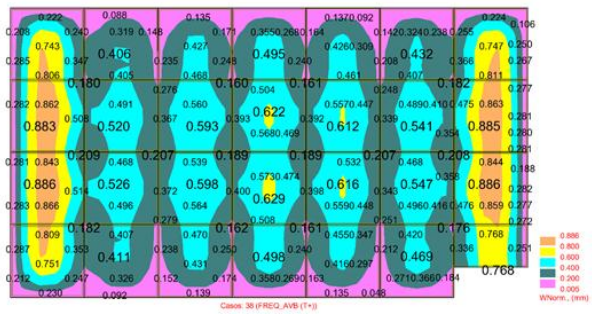
Fig. 22 b) clearly shows the location of the stresses that are superior to the plaster's tensile strength (0,5 MPa), forming a similar cracking pattern to the one present in Fig. 22 a).

Other causes may have also contributed for the cracking, such as:

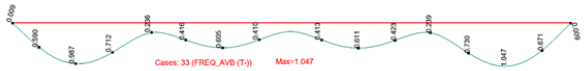
a) Shrinkage of the cement mortar used in the coating layer of the slab in the 2008 intervention;
b) Multiple expansion and contraction cycles of the materials due to thermal and humidity variations, intensified by the dark colored finishing of the slab;
c) Accelerated aging of the mortar due to exposure to the natural elements such as wind, humidity, solar radiation, temperature and frost.

2) Deflection control

Deflection control for the beams and slab was performed limiting the long-term deflection to a maximum of span/400 (L/400). Figure 23 presents the maximum deflection obtained for the slab (3,10 mm) and beams (4,92 mm). Both proved to be within standard limits (L/400=12,83 mm for the slab and L/400=12,50 mm for the beams). It was concluded that the overload placed on the top slab in 2008 did not compromise the deflections limits.



a) Maximum deflection on the slab [mm]



b) Maximum deflection on the beams 0,20x0,50 m ²[mm]

Figure 23. Maximum deflections on the slab and beams

C. Hydrodynamic Pressure and Sloshing

The additional forces caused by the dynamic oscillation of the tank are presented in TABLE VIII.

TABLE VIII. HYDRODYNAMIC PRESSURE VARIATION

Vertical distance (Z) [m]	Hydrodynamic pressure (p_i) [kN/m²]	
	Type 1 Earthquake	Type 2 Earthquake
0,00	0,00	0,00
0,50	0,65	0,39
1,00	1,20	0,72
1,50	1,65	0,99
2,00	2,00	1,20
2,50	2,25	1,36
3,00	2,40	1,45
3,50	2,45	1,48

The maximum sloshing wave height obtain was of 0,32 m for the type 2 earthquake. Since the tank has an interior height of 4,10 m and a maximum height of water of 3,50 m leaving a free height of 0,60 m to the top slab, it is not expected that the sloshing wave would interfere with the top slab.

V. CONCLUSIONS

This project was conducted to assess the structural safety of a water tank, supported by a non-destructive test campaign and a numerical simulation that allowed to evaluate the safety level and to support the definition of the structural retrofitting and rehabilitation needs.

No important damages were detected in the structural elements inspected. However, cracking was observed of the top slab that allows the infiltration of rainwater into the tank together with an insufficient slope for water drainage and an elevated degradation of the waterproofing membrane. Some nonstructural damages were also observed, namely cracking in the non-structural vertical elements, and presence of vegetation in some areas.

Considering the results obtained in the material characterization, it was assumed the concrete strength class C25/30 (f_{cm}=33 MPa) for the structural analysis carried out by numerical modeling.

The environmental vibration test allowed to get the three first natural frequencies (27,1 Hz, 31,0 Hz and 46,7 Hz), and the static load test allowed to estimate the bending stiffness and to verify that the slab was in good structural condition. These results were useful and strongly supported the performed numerical analysis.

The structural safety was generally checked, both for vertical actions and for horizontal actions.

The only exception is the cracking pattern exhibited by the columns that joint the slab in its contour, explained by the forces on these elements for the fundamental combinations associated with the seasonal variation of temperature. The same type of action also explains the cracking pattern observed in the cover slab.

ACKNOWLEDGMENT

The authors would like to thank to Be Water, S.A. (Águas de Ourém) for the support given to this work, namely to Eng. Nuno Martinho and the staff involved during the *in-situ* test.

REFERENCES

[1] NP EN 12504-2, Testing concrete in structures. Non-destructive testing. Determination of rebound number, 2012.

[2] ASTM C803/C803M, Standard Test Method for Penetration Resistance of Hardened Concrete, Book of Standards, Vol. 04.02, American Society for Testing and Materials, 2010.

[3] EN 12504–1, Testing Concrete in Structures- Part 1: Cored Specimens - Taking, Examining and Testing in Compression, CEN, 2009

[4] H. Rodrigues, P. Antunes, A. Costa, P. André and H. Varum, - Chapter 5 "Dynamic measurements as support to the analysis and condition assessment of structures" - BOOK "Accelerometers: Principles, Structure and Applications" - Editors: Paulo André and Humberto Varum, Nova Science Publishers, Inc, ISBN: 978-1-62808-111-4.

[5] P. Antunes, R. Travanca, H. Rodrigues, J. Melo, J. Jara, H. Varum, P. André, "Dynamic structural health monitoring of slender structures using optical sensors," *Sensors*, vol. 12, no. 5, May 2012, pp. 6629-6644, 2012.

[6] RBA, Regulamento do Betão Armado, Decreto-lei nº25 948, de 16 de outubro, 1935.

[7] REBA, Regulamento de Estruturas de Betão Armado, Decreto-lei nº47 723, de 20 de maio, 1967.

[8] RSA, Regulamento de Segurança e Acções em Estruturas de Edifícios e Pontes, Decreto-Lei nº235/85, de 31 de Maio de 1983.

[9] REBAP, Regulamento de Estruturas de Betão Armado e Pré-Esforçado, Decreto-lei nº349-C/93, de 30 de Julho de 1983.

[10] EN 1998-1, Eurocode 8: Design of structures for earthquake resistance -Part 1: General rules, seismic actions and rules for buildings, CEN, 2004

[11] EN 1998-5, Eurocode 8: Design of structures for earthquake resistance Part 5: Foundations, retaining structures and geotechnical aspects, CEN, 2004

[12] G. W. Housner, "Dynamic pressures on accelerated fluid containers," *Bulletin of the Seismological Society of America*, vol. 47, no. 1, pp.15-37, January 1957.

[13] G. W. Housner, "The dynamic behavior of water tanks," *Bulletin of the Seismological Society of America*, vol. 53, no. 2, pp. 381-387, February 1963.

João Veludo is an Adjunct Professor at the School of Technology and Management, Polytechnic Institute of Leiria, Portugal, teaching several topics related with Structural Design and Rehabilitation, and Geotechnical Design. His major research interests are Concrete Building Design and Rehabilitation, Structural Health Monitoring and Foundations Design.

Nuno Almeida is: Civil Engineer, master student and researcher at the School of Technology and Management, Polytechnic Institute of Leiria, Portugal. His major research interests are Structural Analysis, Structural Strengthening and Retrofitting and Building Rehabilitation.

Paulo Fernandes is Coordinating Professor at the School of Technology and Management, Polytechnic Institute of Leiria, Portugal, teaching Reinforced Concrete Structures, Prestressed Concrete and Precast Structures. His major research interests are high performance concrete (HSC, SCC, SFRC, SCSFRC), time dependent behavior of concrete, prestressed and reinforced concrete structures (design, assessment and structural rehabilitation), supported by both experimental and numerical programs.

João Baptista is: Civil Engineer, PhD student at IST- University of Lisbon, Portugal, and researcher at the School of Technology and Management, Polytechnic Institute of Leiria, Portugal. His major research interests are Experimental Testing, Structural Analysis and Concrete Technology.

Hugo Rodrigues is Senior Lecturer at the School of Technology and Management, Polytechnic Institute of Leiria, Portugal, teaching several topics related with Structural Analysis, Building Pathology and Rehabilitation. His major research interests are Building Rehabilitation, Structural Health Monitoring and Seismic Safety including experimental and numerical activities.

Nuno Martinho is Technical Director at Be Water S.A., Ourém, Portugal, with more than 10 years of experience specializing in asset management and water network systems operations and maintenance.

Florindo Gaspar is a Senior Lecturer at the School of Technology and Management, Polytechnic Institute of Leiria, Portugal. His research interests are mainly related with the study of materials and construction processes, and inspection and evaluation of structures.