Structural Safety Analysis of the Aqueducts "Coll De Foix" and "Capdevila" of the Canal of Aragon and Catalonia

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Summary

The Canal of Aragon and Catalonia (CAC) is 134 km long and irrigates 105,000 ha (131 irrigation user communities) and it is own by the River Ebro's Water Agency. The aqueducts are located between km 67 and 71 of the canal and were designed by the Civil Engineer Félix de los Ríos Martín in 1907. The cross-section of both aqueducts, Coll de Foix and Capdevila, was extended within the framework of the project by Fernando Hué Herrero in 1962 in order to reach design flows of 26.1 m^3/s and 25.7 m^3/s, respectively. The structural performance of the aqueducts has been satisfactory; nevertheless, the hydraulic capacity has reduced over the years. As a result, the irrigation user communities have expressed the need to extend the cross-section of the aqueducts to meet the irrigation demands. Given the age of the structure and the different design considerations at the time, it is paramount to verify the structural reliability of the aqueducts in the new load configuration. Therefore, the objective of the paper is to present the structural safety analysis conducted and describe the new extended cross-section for both aqueducts (maintaining the original structural typology).

Keywords: heritage; maintenance; reinforced concrete; safety

1. INTRODUCTION

The Canal of Aragon and Catalonia (CAyC) is a hydraulic infrastructure of national interest built in 1906. The canal collects water from the Esera river and it is used for agricultural purposes, water supply in urban areas and industries and hydroelectric energy generation.

Water regulation and usage has significantly changed since the beginning of the 20th century and, consequently, the canal has been subjected to refurbishment to improve coatings and increase the cross-section of the main canal and irrigation ditches. The aqueducts assessed in this study are an example of this evolution and refurbishment.

The Interministerial Commission of Hydraulic Plans agreed in 1960 on the construction of the Liaison Canal that complements the water supply of the Esera river with water from the adjacent hydraulic basin, the Noguera- Ribagorzana. This Liaison Canal covers 44,000 Ha, has a capacity of 26.1 m^3/s and connects with the CAyC in the km 66. Due to the connection of the two canals, an extension downstream of this point was required to increase the capacity of the latter (CAyC) whose capacity was limited to 12.42 m^3/s.

Two aqueducts downstream the connection point, Coll de Foix Aqueduct (Figure 1) and Capdevila Aqueduct (Figure 2) present design flows of 26.1 m^3/s and 25.7 m^3/s, respectively. These design flows do not correspond with the current real capacity, which approaches 21.9 m^3/s in Coll de Foix. Under these circumstances, the need to increase the hydraulic capacity of both aqueducts is mandatory to maximise its usage.

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Figure 1. Coll de Foix Aqueduct located in Km 67 of CAyC

Figure 2. Capdevila Aqueduct located in Km 71 of CAyC

Currently, an expansion of the section of the canal is envisaged to tackle the loss of hydraulic capacity due to: (1) a roughness coefficient different from that foreseen in the original project; (2) a variation of the hydraulic slope downstream of both aqueducts as a result of the changes in the grade of the aqueduct and, (3) an increase in the water demands of the users, which translates into the need to increase the circulating flow.

The objective of this paper is to raise and analyse the structural feasibility of expanding the sections in order to increase their hydraulic capacity. For this purpose, a survey of damages, topographic and geotechnical studies and structural analysis with non-linear models has been carried out. Fortunately, the original project of D. Félix de los Ríos Martín [1] and of two projects of D. Fernando Hué Herrero [2-3] is available. The technical and scientific interest of this work lies in the study of a concrete structure with more than 100 years in service, designed with different calculation methods, and whose results will serve as a reference for interventions in similar works. The global project can be found in [4].

2. GEOMETRY AND MATERIALS

2.1 Vaults and spandrels

The Coll de Foix aqueduct is composed of a line of 7 elliptical arches spanning 8.50 m reduced to ¼, comprised between 6 piles and 2 abutments that longitudinally delimit the aqueduct to 69.70 m. On the

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other hand, the Capdevila aqueduct is 79.20 m long and has 8 elliptical arches spanning 8.30 m and equally reduced to ¼ of the span.
The arches are made of plain concrete composed of 250 kg/m^3 of slow setting Portland cement. The thickness of the arches in the key is 0.50 m and increases progressively to a thickness of 0.70 m. The compressive strength values obtained from the cores extracted from the aqueduct arches of Coll de Foix and Capdevila are 42 N/mm^2 and 22 N/mm^2, respectively.

The spandrel walls are composed of pressed brick, breaking continuity between contiguous arches through projecting pilasters that form the abutments. The arrangement of reduced elliptical arches leads to reduced spandrel walls surfaces, thus making a backfill of concrete with 100 kg/m^3 of natural cement economically viable. This alternative to the classic compacted granular filler allows to minimize the risk of leaks due to water losses in the slab, which would increase the lateral pressures in the spandrel walls. In turn, the fact that both have similar stiffnesses is favourable for the arch-filler interaction.

2.2 Piers and foundations

The vertical piers are pyramid-shaped to increase stability, with 4% and 6% inclination laterally and in the fronts formed by the abutments. The height ranges from 1.00 m to 6.20 m starting from the foundations to the start of the arches. At the ends, the aqueducts are supported by abutments founded on soft sandstone and that delimit the aqueducts longitudinally. The filling of the piles and abutments is made of plain concrete with 225 kg/m^3 of natural cement.

The primitive foundations are rectangular and made of concrete with 160 kg/m^3 of natural cement. Although, only Coll de Foix aqueduct maintains the original foundations since they are still referred to the layer of compact clays since its execution. On the other hand, in the Capdevila aqueduct it was necessary to reinforce, on two occasions, the foundation of some of its piers. In 1925, a stack of wooden piles was made only in pier 3 (Figure 3), which proved to be insufficient because it did not reach the competent stratum. In the 1962 project [3] the definitive recesses of piers 1-6 were designed and executed with concrete caissons of sufficient depth to reach the competent stratum of compact clays.

The values of compressive strength obtained from the cores drilled from the foundations are 20 N/mm^2 in both aqueducts.

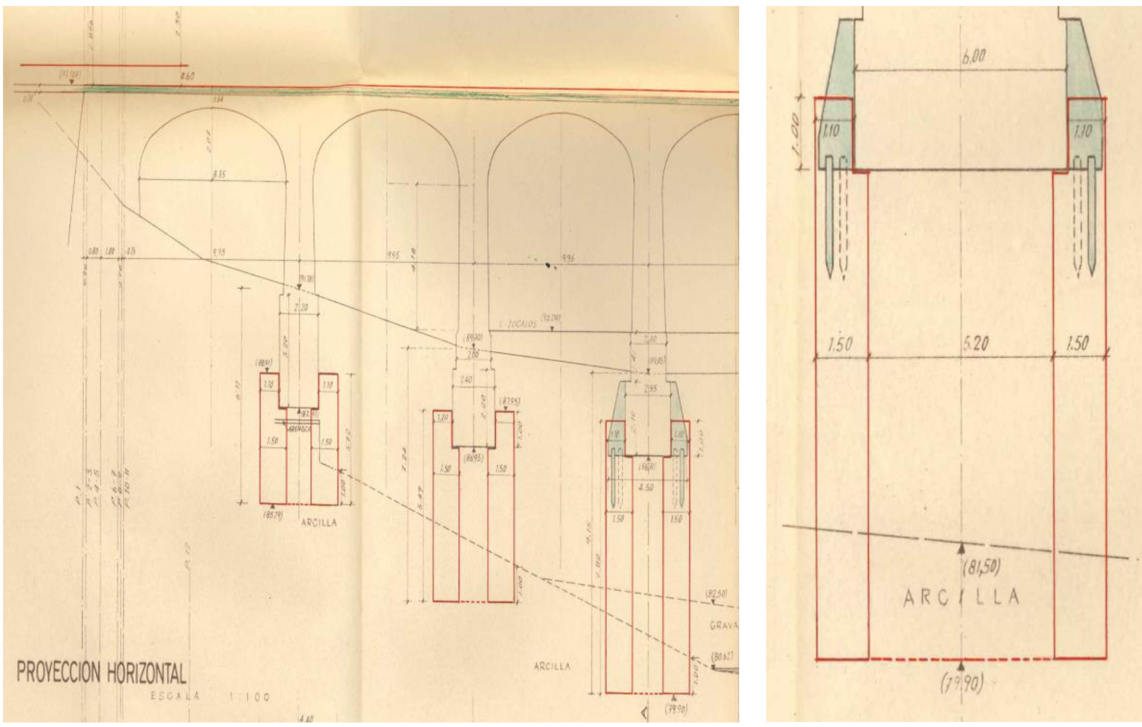

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Figure 3. Capdevila aqueduct in the CAyC in Km 71

2.3 Canal

The section of the canal is constant in both aqueducts (Figure 4) and consists of a slab of 3.00 m wide supported on the back of the arches and joins the cashiers through chamfers of 0.70 m. The cross section is braced on the top by rectangular section braces measuring 0.2x0.2 m^2 each 2.0 m in the longitudinal direction. The original height of the cross section is 2.40 m and the proposal is to increase it 0.60 m. All the elements of the canal have a thickness of 0.20 m.

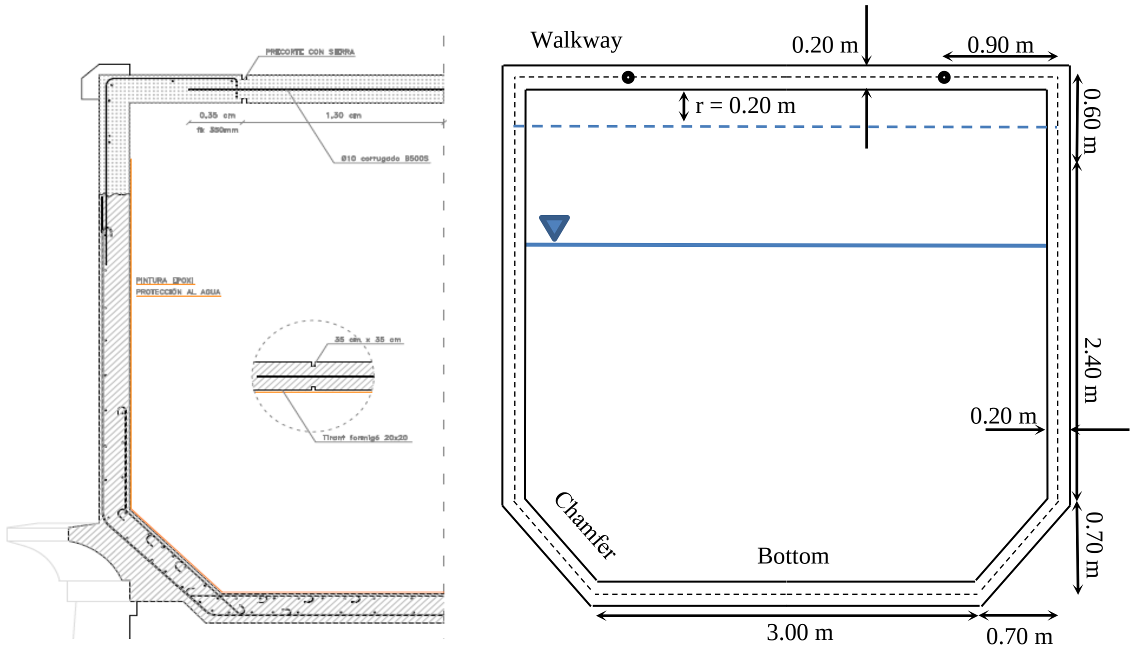

Figure 4. Geometry of the cross section of the canal and reinforcement configuration

On the one hand, the channel in the aqueduct was built with vibrated hydraulic concrete with a content of 300 kg/m^3 of artificial Portland cement. The strengths obtained from cores extracted from the aqueducts of Coll de Foix and Capdevila are 45 N/mm^2 and 30 N/mm^2, respectively. On the other hand, the reinforcement consists of smooth steel bars with a minimum yield stress (f_y) of 240 N/mm^2. The reinforcement of the extension consists of B500 steel rebars (f_{yk} = 500 Mpa) and plastic fibers that are mixed with the concrete (f_{ck} = 30 N/mm^2). The flexural residual strength provided by the fibres are $f_{R1k} \geq 1.50$ N/mm^2 and of $f_{R3k} \geq 1.50$ N/mm^2.

The proposed extension entails an increase of the hydraulic section of 19.1% of the final section (13.1 m^2) with respect to the current one (11.0 m^2) maintaining a clearance (r) of 0.20 m. This extension implies an increase of 8.5% in permanent loads (empty channel) and 10.0% in service (full channel, r = 0.0 m).

3. STRUCTURAL RELIABILITY OF THE EXPANDED CANAL

3.1 Geometry simulation

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A 3D finite element model (Figure 5) was implemented with SAP2000®. A 10.0 m length module of canal (distance between piles) was simulated by using shell elements, except for the concrete ties that were represented with beam elements, and taking into account the dimensions established in Figure 4.

The boundary conditions taken into account were: (1) the connection bottom-chamfer is simulated as a simply supported point with free rotation in the cross-section plane and restrained displacement along the three spatial directions. This condition is representative in view of the existence of a lateral overhang that restricts the displacements. (2) The interaction between the bottom and the spandrel has been simulated by considering that downward displacements are restricted while upwards displacements are free to occur; however, it has been proven that for all load combinations the bottom detaches from the spandrel. (3) The connection between the walkways and ties is represented by means of a perfect plastic hinge.

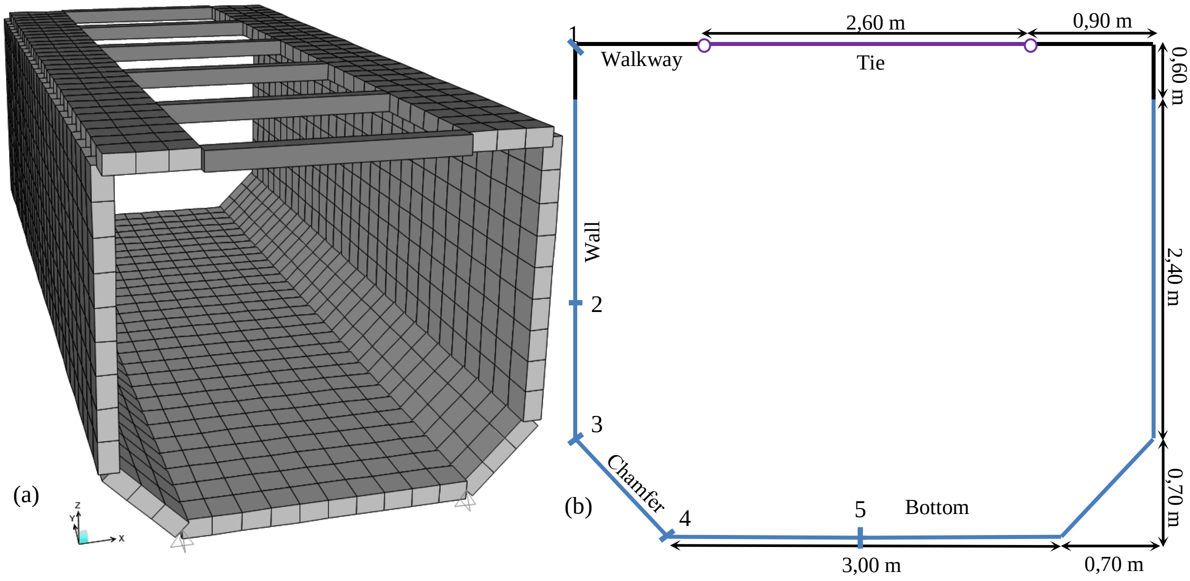

Figure 5. (a) 3D Finite element mesh of a 10.0 m length canal module and (b) control sections (1-5)

3.2 Loads and load combinations

The *loads* considered were: (1) self-weight (*G*). (2) Water circulating through the canal ($Q_{k,1}$). (3) Life load circulating onto the walkways ($Q_{k,2}$) with a magnitude of 5.0 kN/m^2. (4) Wind load applied onto the external lateral surfaces of the walls ($Q_{k,3}$) consisting in a uniformly distributed action with a magnitude of 1.02 kN/m^2 that can be either pressure or suction depending on the wind direction.

The following load combinations were assumed:

- *Service limit state*: a representative combinations of loads of the operational stage of the canal (C_1) and consisting of the gravitational load (*G*) and the water weight and pressure effects ($Q_{k,1}$) with a clearance of r = 0.20 m has been imposed. No partial safety factor has been considered for the loads.
- *Ultimate limit state*: a permanent situation of loads (C_2) formed by the gravitational action (G), the water load ($Q_{k,1}$) with no clearance (r = 0.0 m, hydraulic full cross-sectional capacity); the life load onto the walkways ($Q_{k,2}$) and the wind action ($Q_{k,3}$) were combined and factored with the load partial safety factors defined in the EHE-08 [5]. Additionally, an accidental situation (C_3) has been defined in order to take into account a potential water discharger over the walls (0.50 m of water level over the walkways) due to an operational error of the canal lock-gate. The load partial safety factors defined in the EHE-08 [5] were also applied to establish this load combination.

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3.3 Service limit states

Regarding the service limit state of deformations, the Figure 6 gathers the displacements field for the load combination C_1. The maximum horizontal displacement is 2.0 mm (opening of the hydraulic cross-section) while the maximum upwards displacement of the bottom is 0.1 mm. Both magnitudes are considered to be assumable at structural level and do not imply any esthetical concern since those would not perceptible from the walkways or a car driver circulating below the infrastructure. It is worth to note that the bottom tends to detach from the spandrel.

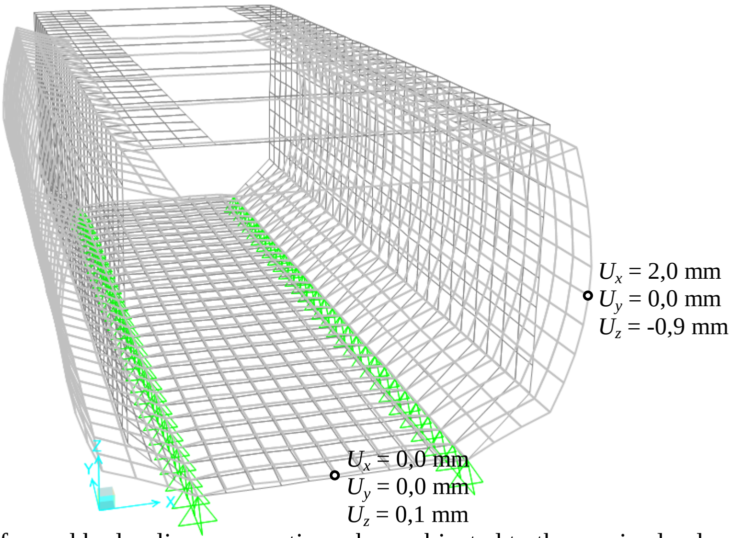

Figure 6. Deformed hydraulic cross-section when subjected to the service load combination (C_1)

Regarding the cracking service limit state, the Figure 7 represents the M_{11} (activate the longitudinal reinforcement, secondary) and M_{22} (activate the transversal reinforcement, main) bending moments distributions for the load combination C_1. The characteristic crack width (w_k) is assessed considering this combination and it has been confirmed the limit state condition $w_k < w_{max}$ = 0.30 mm, where w_{max} is the maximum crack width allowed for the environmental conditions to which the structure is subjected.

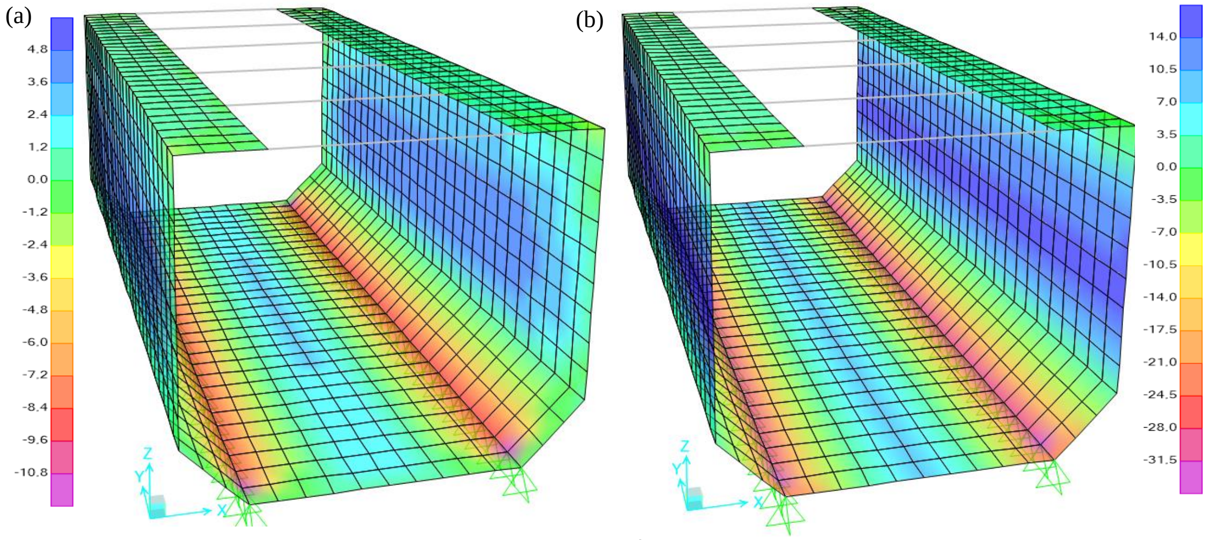

Figure 7. Bending moment distribution (mkN/m) for the combination C_1: (a) M_{11} and (b) M_{22}

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The results gathered in Figure 7 highlight that peak values of M_{22} up to 15.7 mkN/m are reached at the lateral walls while values of -32.8 mkN/m are expected at the chamfer-bottom joint section (control sections 2 and 4, respectively). Likewise, it can be confirmed that the M_{11} bending moments are lower in magnitude respect M_{22} (about a 30% of M_{22}) and these respond to a secondary longitudinal bending induced by the geometry and boundary conditions.

Table 1 reflects the number of 12 mm in diameter steel reinforcing bars Φ12, $n_{\Phi 12}$, the steel reinforcing area (A_s), the average cracking bending moment strength (M_{crm}) and the w_k assessed for every control cross-section (Figure 5b) when subjected to M_{22}. It can be confirmed that only the control section 4 is expected to crack ($M_{22} \geq M_{crm}$), w_k being inferior to 0.10 mm in this section.

Table 1. Cracking service limit state and bending ultimate limit state checks for the control sections

			Cracking service limit state (C_1)			Ultimate limit state				
							Permanent (C_2)		Accidental (C_3)	
Section	$n_{\Phi 12}$	A_s (mm^2)	M_{crm} (mkN/m)	M_{22} (mkN/m)	w_k (mm)	$M_{22,u}$ (mkN/m)	$M_{22,d}$ (mkN/m)	$FS_{u,M}$	$M_{22,d}$ (mkN/m)	$FS_{u,M}$
1	4	452	23,9	-1,6	-	26,4	4,9	5,4	2,2	12,0
2	8	905	23,9	15,7	-	32,7	29,1	1,1	31,3	1,0
3	8	905	23,9	2,1	-	32,7	21,6	1,5	10,5	3,1
4	24	2714	-23,9	-32,8	0,10	-90,6	-56,9	1,6	-54,5	1,7
5	8	905	23,9	8,4	-	32,7	8,4	3,9	5,3	6,2

The ties are only subjected to pure traction forces (T) since the connection to the walkways is solved with plastic hinges (no bending forces are transmitted). The maximum magnitude of T is 20.7 kN, the average cracking tensile force (T_{crm}) being 102.4 kN (the average tensile strength, f_{ctm}, considered is 2.56 N/mm^2). Consequently, the likelihood of cracking is low and no cracks are expected due to direct loads. Nevertheless, taking into account that the ties are critical elements for the safe and suitable structural performance of the hydraulic section under service conditions, these were considered as cracked for the cracking service limit state verifications. Even with this safe-side assumption, the value of w_k expected to be reached during operational conditions is 0.12 and, thus, inferior to the 0.30 mm allowed for this structure to guarantee that the reinforcement will not suffer from corrosion.

3.4 Ultimate limit states

Figure 8 gathers the M_{22} bending moment envelopes for the combination C_2 ($M_{22,d}$). $M_{11,d}$ envelope is omitted since this is not determining for the reinforcement design; however, the pattern is similar (with different magnitudes) to the one presented in Figure 7a. For the accidental load combination (C_3), the bending moment envelopes' pattern also respond to those presented in Figure 8. In Table 1 the ultimate limit state checks are reported.

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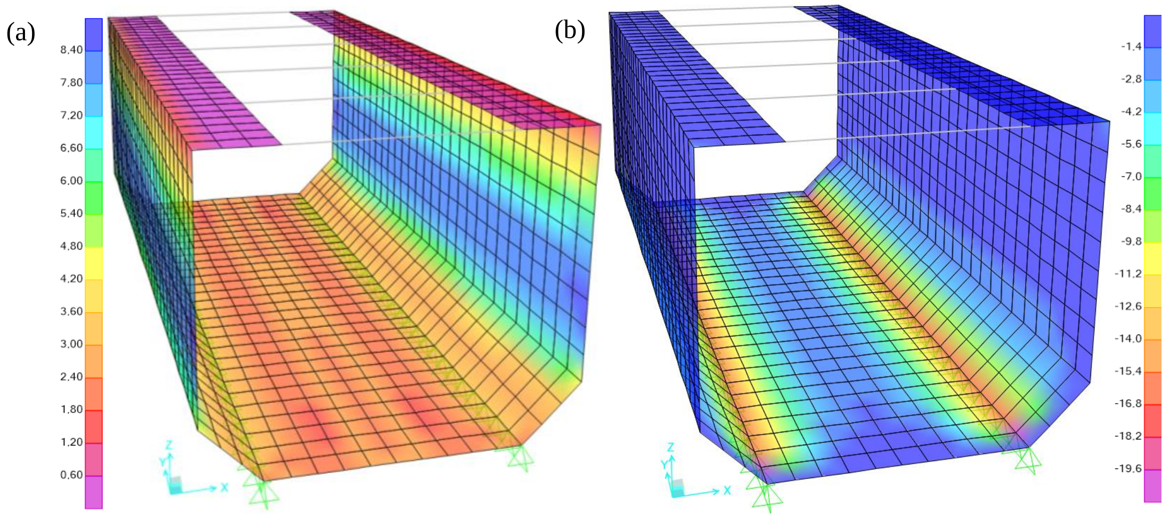

Figure 8. Bending moment envelopes (mkN/m) for the combination C2: (a) $M_{11,d}^{max}$ and (b) $M_{11,d}^{min}$

The results gathered in Table 1 confirm that the global safety factor against bending moment in ULS, defined as $SF_{u,M} = M_u/M_d$ is higher than 1.0 and, hence, the hydraulic cross-section is structurally reliable. Likewise, the ties are also safe in ULS since $SF_{u,T} = T_u/T_d$ is 2.0 and 1.1 for the load combinations C_2 and C_3, respectively.
Finally, the shear envelopes (V_d) in ULS for the load combination C_2 are presented in Figure 9. It must also be remarked that the envelopes follow the same pattern for the load combination C_3.

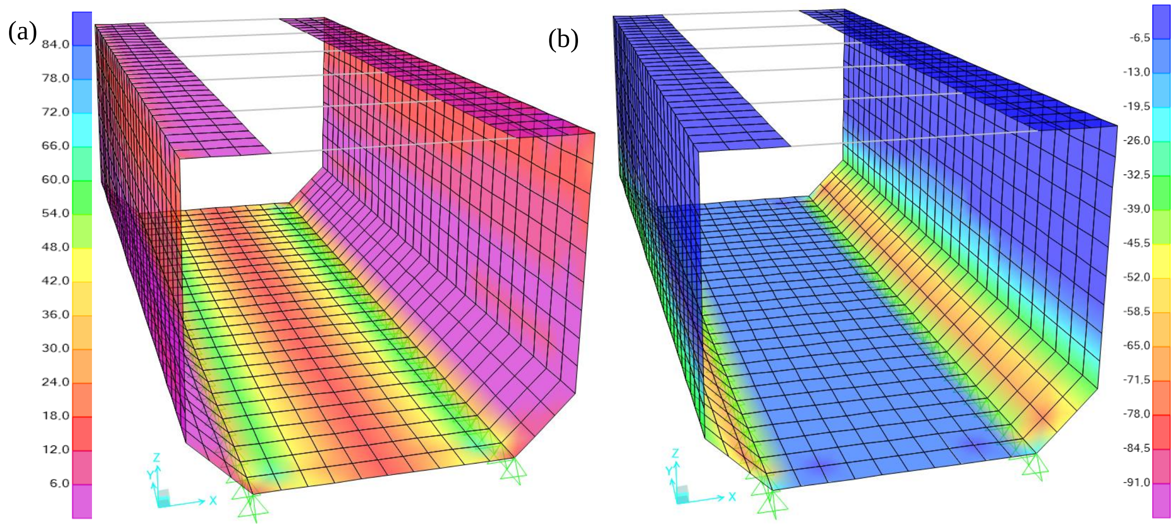

Figure 9. Shear envelopes (kN/m) in ULS for the load combination C_2: (a) V_d^{max} and (b) V_d^{min}

The global safety factor against shear $SF_{u,V} = V_u/V_d$ is defined, V_u (=127.3 kN/m) being the shear strength of the cracked cross-section subjected to tensile stresses in ultimate limit state. The máximum values of V_d are 72.4 Kn (C_2) and 72.2 Kn (C_3) and, consequently, $SF_{u,V} > 1{,}00$ for the hydraulic cross-section.

4. CONCLUSION

After a thorough structural analysis of both aqueducts using the safety format accepted into the Spanish Structural Concrete Code (EHE-08) it has been confirmed that the lateral walls can be increased up to 0.60 m. This will lead to an increase of 19.1% of the hydraulic capacity while keeping the total weight increase below 10%. These type of analyses could serve as guides for future similar

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designs (in Spain there are dozens of similar aqueducts that are requiring repairs and extensions of the hydraulic capacity).

5. REFERENCES

[1] de los Ríos, F., Coll de Foix Aqueduct Project, 1907.
[2] Hué, F., Consolidation and covering of the Capdevila Aqueduct, 1949.
[3] Hué, F., Project for the increase of the hydraulic capacity of the Aragón and Catalonia Canal, 1961.
[4] de la Fuente, A., Aguado, A., Hydraulic capacity increase for the Coll de Foix and Capdevila Aqueducts, 2016.
[5] CPH 2008. *Structural Concrete Spanish Guide*. Ministerio de Fomento, 2008.