Structural design, modelling, material testing and construction of a new stone masonry arch bridge in Vila Fria, Portugal

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ABSTRACT: This paper presents the main activities involved in the construction of a new stone masonry arch bridge, the Vila Fria Bridge located in the north of Portugal, as part of a wider multidisciplinary project. The paper briefly describes the main aspects related to the design, construction, the characterization of the adopted materials and the numerical simulations to support the undertaken choices from the construction to the beginning of the bridge service conditions.

1 INTRODUCTION

Although referring to typically ancient constructions, the research about the structural behaviour of stone masonry bridges has become again a matter of concern. The fact that a large number of such type of durable bridges exist, quite often subject to heavier loads than those they were supposed to sustain, has been drawing increased attention of the technical community to their structural safety and maintenance conditions. Therefore, any opportunity based on experimental evidence obtained from real masonry arch bridges is generally of great interest as it may contribute for improving the knowledge on their structural behaviour and performance.

A new stone masonry arch bridge was recently built in Vila Fria, Felgueiras, Portugal. Following a direct invitation, the Institute of the Construction (IC) of the Faculty of Engineering of the University of Porto (FEUP), through one of FEUP laboratories, the Laboratory for Earthquake and Structural Engineering (LESE), accepted to collaborate in this initiative, in the framework of a multidisciplinary scientific work under development concerning the structural analysis, monitoring and assessment of stone masonry bridges.

In this context, an overall description of the bridge and its particular aspects is herein addressed, starting from the basic design criteria and the various construction phases. For the design, classical methods based on lines of thrust were adopted and further supported by more refined analyses. Concerning the construction, traditional processes were adopted for the piers, abutments, arches, spandrels, backfill and secondary elements.

Several experimental tests (laboratory and "*in situ*" tests) were carried out with the purpose of collecting a good characterization of the materials used in the construction of the new Vila Fria Bridge, particularly those considered to have significant influence in the structural behaviour of the bridge, namely the stone, the infill and interface materials between different elements.

Finally, in order to support the choices undertaken since the design phase until the bridge service beginning, detailed structural analyses were carried out resorting to a sufficiently refined finite element model with suitable element discretization and modelling techniques.

2 BRIDGE DESIGN

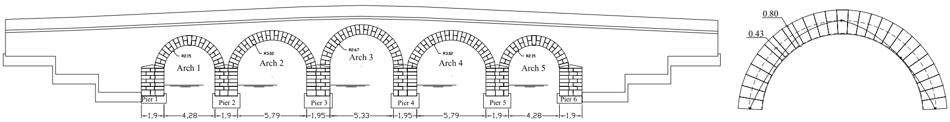
The new bridge design took into account several criteria and particular aspects of the bridge location. As for any other bridge, the guidelines were the local topography, the environmental aspects (a rural area, in this case) and the common topologies of stone arch bridges.

In addition, the specific requirement from the local (Felgueiras) municipality stated that the bridge should be a granite stone masonry arch bridge, respecting (as much as possible) the existing piers, and the deck width should be proportioned to allow two-way traffic.

Considering the specific architectural requirements, the solution shown in Fig.1a (Costa, A. *et al*. 2001) was finally proposed. It consists of a bridge materialized by five arches, having 4.8 to 6 m of span, with four piers and two abutments supporting a two-ramp deck 60 m long (total length) and 6 m wide.

The dimensions of bridge components were defined by geometrical empirical relations given from a set of observations of stone masonry arch bridges by Lagomarsino, S. *et al.* (1999) and Gambarotta, L. (1999) and also inspired in similar constructions in the North of Portugal (Costa, C. et al. 2005).

The safety of the arches was verified applying the safe theorem of the plastic analysis (Heyman, J. 1995), considering essentially two load cases: case 1, includes the dead loads (of arches, spandrel walls and backfill) and live loads consisting of a uniformly distributed load on the deck (3 kN/m^2) and a "line load" (30 kN/m) applied in most unfavourable sections (at ¼ span or near the keystone); case 2, consists of the dead loads and the "standard vehicle" corresponding to class II bridges (three axle load, 100kN per axle) as defined in the Portuguese code standard for safety and actions RSA (1985), located also in the most unfavourable sections. According to the adopted methodology, loads were not affected by partial safety factors and a minimum geometrical factor of safety of 1.9 was reached for case 2 (Fig.1b), the most conditioning one. This factor is defined as the relation between the real arch thickness and the minimum thickness that is strictly necessary to satisfy the static equilibrium of the arch; the obtained value fits well the suggestion that values around 2.0 are suitable for safety purposes (Heyman, J. 1999).



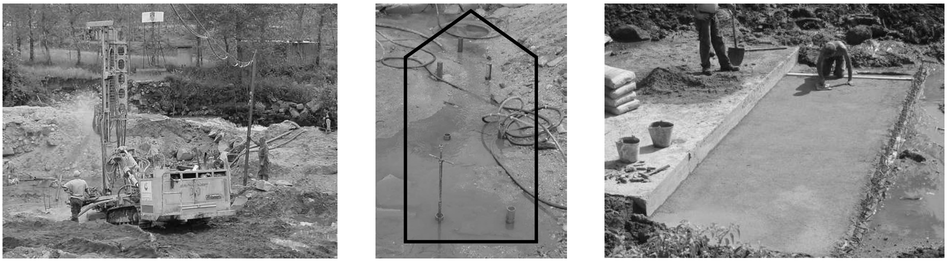
a) b)

Figure 1 : The new Vila Fria Bridge. Upstream view (a). Thrust line of arch 3 (b).

3 CONSTRUCTION

3.1 *Foundations*

Due to extremely bad soil conditions, the new Vila Fria Bridge was built on deep foundations made by groups of micro-piles (Figs.2a,b) with reinforced concrete top caps (Fig.2c) on which the bridge stone piers were mounted.



a) b) c)

Figure 2 : Bridge foundations – (a) Micropile instalation, (b) micropiles and (c) top RC cap.

3.2 Bridge structure

Supported by the micro-pile RC caps, the solid piers were built in massive stone layers separated by beds and vertical joints comprising a thin layer of weak mortar (see Fig. 3a). This mortar is especially produced for this type of applications and consists on a mixture of lime based mortar with a small percentage of Portland cement.

In the subsequent phase, the temporary wood centering for each arch was placed directly supported in the ground after having deviated the river towards other lateral flow zones. In order to resist the arch weight before its final closed configuration, the centering was made of several strong wood frames giving support to thick wood boards on which the arch stones were laid (Fig. 3b).


a) b)

Figure 3 : Vila Fria bridge structure – (a) piers and (b) arches

The stone assembly of the main elements (piers, arches and breakwaters) was already specified in the design in terms of their dimensions and construction sequence. The subsequent step consisted on the elevation of spandrel walls and posterior spread of the backfill material (*tout-venant*) along the bridge, as can be seen in Fig. 4a.

Above the backfill material, a layer of *tout-venant* mixed with cement provided a regularized surface covered by a waterproof membrane. A final layer of a dry sand and cement mixture was made in order to lay down the granite stone pavement, as shown in Fig. 4b. Parapets are also made of granite stones fixed to the spandrel walls by means of two stainless steel bars. Fig. 5 shows the final result after three years of bridge construction opened to traffic by May 2005.


a) b)

Figure 4 : Vila Fria bridge structure – (a) backfill material and (b) granite stone pavement

Figure 5: The new Vila Fria Bridge. Aerial view. Courtesy of Prof. F. Piqueiro and Foto Engenho ©.

After bridge completion, the monitoring tests developed in two complementary ways: evaluation of short time response of the bridge (during a specific load test under service conditions) and long term monitoring of the bridge subjected to environmental loads and traffic loads also under normal service conditions. These issues are discussed in a companion paper (Arêde, A. *et al.* 2007).

However, it is worth mentioning that during all the construction phases, several tasks related with the monitoring project had to be made compatible with the remaining works. The main concern was related to the high sensitivity of some sensors and to the large number of electric and optical cables that should be carefully installed and protected during the construction. For that purpose a longitudinal technical gutter was made along the downstream of the bridge, right under the road surface and near the parapet, where all cables are collected and directed to technical cabinet built in the north side of the bridge.

4 PHYSICAL AND MECHANICAL CHARACTERIZATION OF THE MATERIALS

In order to characterize the materials used in the construction of the Vila Fria Bridge, both laboratory and *in situ* tests were carried out. The stone was tested in the Construction Materials Laboratory (LEMC) of FEUP, whereas the backfill material was tested both *in situ* and in laboratory by the staff and facilities of the Geothecnics Laboratory (GL) of FEUP; shear tests over mortared joints, dry joints and joints between blocks and infill material were also performed by recourse to specific equipment of rock mechanics belonging to the Civil Engineering Department (DEC) of FEUP.

4.1 Stone characterization

Mechanical and physical characterization of the stone resorted to the following tests, done in accordance with the referred standards: uniaxial compressive strength on natural moisture conditions (NP EN 1926 – 2000); uniaxial compressive strength on dry and saturated samples (NP EN 1926 – 2000); splitting tensile strength by the Brazilian method (ASTM-D3967-95a – 1995); Young modulus (ASTM-D3148-02 – 2002); porosity of natural stone (DIN-52-102 - 1988); capillary for water absorption coefficient estimation (NP-EN1925 – 2000) and freeze-thaw cyclic tests (DIN-52-104 – 1982). The results from these tests are described in Table 1.

Concerning the mechanical strength, the results show that the stone is compatible with the bridge requirements, despite the significant reduction of strength due to possible saturation of stone; however, bearing in mind that very low compressive stresses are expected in this type of constructions (just around a few MPa), a significant safety factor is still available for compressive strength. The value for the modulus of elasticity may be considered low; however, these results have to be seen with some reserves, mainly because there are still some doubts about the adopted setup for this test. For that reason, a new campaign of deformability tests is about to be performed, in order to confirm or improve the above listed value.

Nevertheless, the porosity and water absorption coefficient from capillary is considered high for this type of stone, indicating some meteorization, with the global porosity network well interconnected. This fact may rise some concern in terms of durability aspects.

The results from the freeze-thaw tests allowed the following conclusions: (*i*) the visual inspection and the weight of the specimens did not indicate any mass loss or degradation on the physical properties of the stone; however, (*ii*) the compressive strength reduced significantly among the dry samples (-27%), while this parameter remained almost unchanged in the saturated samples (+2%).

4.2 Characterization of the backfill material

In order to evaluate the physical and mechanical properties of the backfill material, both laboratory and *in situ* tests were made by recourse to *radioactive cell*, *sand bottle*, *proctor* and *triaxial* equipment. Table 1 summarizes the results from these tests (except for the *triaxial test*).

Table 1 : Test results.

a) Stone tests results (mean values)

Compressive strength			Tensile strength	Young modulus	Porosity	Water absorption coef.
Natural	Dry	Saturated				
MPa	MPa	MPa	MPa	GPa	%	$g/m^2/s^{-0.5}$
64	66	32	4	35	4	37

b) Backfill tests results (mean values).

	Radioactive cell	Sand bottle	Proctor
Density (kN/m^3)	17.13	19.20	21.5
Water coefficient (%)	6.65	7.20	5.2

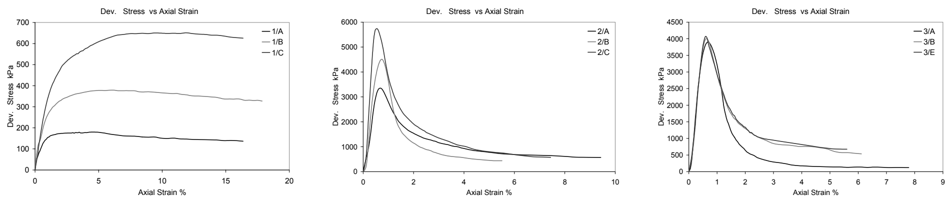
c) Specimen identification tested on the triaxial camera

Material type		*Tout-venant*			*Tout-venant* and cement					
Specimen		A1	B1	C1	A2	B2	C2	A3	B3	C3
Specimen diameter	(mm)	150	150	150	150	150	100	100	100	100
Cement	(%)	0	0	0	10	10	10	7	7	7
Vertical stress	(kPa)	30	80	150	50	50	50	10	50	80
Consolidation time	(days)	-	-	-	4	8	28	28	28	28

From these tests, the first conclusion is that a good homogeneity can be found along the bridge; this fact is supported by the comparison of values among all the tested samples.

The results in *proctor* tests showed an average compaction level of 92%, which can be considered a good level of compaction, when compared to the limits imposed in the good practice of road construction.

In order to characterize the shear resistance of the backfill material by recourse to the *triaxial* test, nine specimens were tested. Table 1 identifies the specimens and describes the parameters changed between them. Fig. 6a,b,c shows the results from *triaxial* tests in specimens of *tout-venant* and *tout-venant* mixed with cement in two different proportions.



a) b) c)

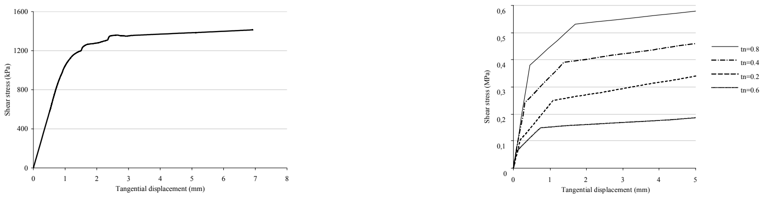
Figure 6 : Stress-strain results from triaxial tests: (a) *tout-venant*, (b) *tout-venant* with 10% cement and (c) *tout-venant* with 7% cement

Concerning the behaviour shown in the three curves of Fig. 6a,b,c (note the different scales!) the results show typical elastic-plastic soil behaviour for the case without any cement. When cement is added, the curve reaches a peak strength typical of cohesive materials, followed by a softening zone towards a residual value. These results will be particular relevance for numerical modelling of the backfill material.

4.3 Joints

In order to describe the behaviour of the joints between different elements of the bridge, shear tests were carried out using equipment of DEC-FEUP.

The shear strength of masonry joints is evaluated by monitoring the sliding along their discontinuity plan. Tangential displacements are imposed under a given constant normal force, and both the corresponding tangential forces and the normal displacements are measured. This procedure was iterated for different values of the normal compression stress varying from 0.2 to 2 MPa in several specimens consisting of: *i)* two blocks and a thin layer of mortar in the interface (case 1-mortared interfaces); *ii)* two blocks without any mortar in the interface (case 2-dry interfaces) and *iii)* one stone block and infill material (case 3-block/*tout-venant* interfaces). Fig. 7a shows the result of one shear test in mortared interfaces and Fig. 7b depicts the theoretical fitted curves as used in the numerical model for monotonic behaviour. Further tests are still under development, for both cyclic shear and normal behaviour characterization.



a) b)

Figure 7 : Shear behaviour for mortared interfaces. (a) Experimental and (b) theoretical fitted curve.

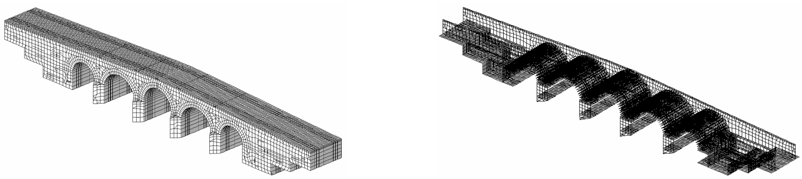
5 NUMERICAL SIMULATION

5.1 Description of the numerical model

For additional support to design options the structural analysis of the Vila Fria bridge was carried out using a 3D structural numerical model using the finite element method by means of CAST3M software (Pasquet, P. 2003). The stone blocks were simulated by solid elements with linear behaviour duly individualized, in order to consider explicitly the following joints: *i)* in the arches zone; *ii)* between the piers and the remaining structure; *iii)* between the infill and the spandrel walls and arches; *iv)* between the breakwaters and the remaining structure. The deck zone was also discretized with 3D elements. All the interfaces between the distinct components of the structure were simulated using joint elements allowing the nonlinear behaviour to be activated. Fig. 8a,b shows the whole meshes of both solid and joint elements for the Vila Fria bridge finite element model.

Since the bridge consists of a new structure the geometrical characterization was based in the design drawings. The mechanical parameters were based in laboratory tests, using samples of the used materials in the construction, and "*in situ*" tests, as described previously, and in results of preceding studies on the same kind of materials.

The calibration of this numerical model, as well as similar studies on other masonry arch bridges (Costa, C. 2002), is still underway (but soon available) through the result comparison of numerical analysis and experimental evidence obtained from ambient vibration tests and from load tests on the bridge as those reported in a companion paper (Arêde, A. *et al.* 2007).



a) b)

Figure 8: Finite element mesh of (a) 3D elements and (b) joint elements used in the numerical analysis.

The volumetric elements of density (ρ) were considered with linear elastic behaviour controlled in terms of Young modulus (E) and Poisson ratio (ν). The behaviour of joint elements is controlled through the normal and tangential contact stresses and the corresponding relative displacements between the two joint faces (opening/closing and slipping of the interface between the blocks) with a Coulomb frictional non-linear model without dilatancy (Pegon, P. and Pinto, A.V. 1996) available in CAST3M software.

The mechanical properties of the materials used in the numerical model are shown in Table 2 in terms of Young modulus (E) and density (ρ) for the blocks and in terms of initial normal and tangential stiffness (k_n and k_s, respectively) for the joints. The non-linear behaviour is characterized by recourse to the experimentally obtained curves as above described.

Table 2 : Material parameters for linear elastic analysis.

Blocks				Joints		
Zones	E (GPa)	ρ (kN/m^3)	ν	Zones	k_n (MPa/mm)	k_s (MPa/mm)
Load degradation slab & foundations	10	25	0.2	Main bridge / breakwaters	6.241	0.681
Deck	2.1	25	0.2	Infill / arches and piers	0.032	0.004
Infill	1.5	21	0.2	Infill / spandrel walls	0.032	0.004
Spandrel walls & breakwaters	6.5	26	0.2	Arch's blocks	6.241	0.678
Arches & piers	35	26	0.2	Pier's blocks	6.241	6.781

5.2 *Result analysis*

For the numerical analysis the same load cases 1 (dead load + live uniformly distributed and linear loads) and 2 (dead load + standard vehicle live load) were considered as already described for the design process. In addition, another load case consisting only of the dead loads was also considered for comparative purposes. The simulation of traffic loads was based on the application of point loads located in several positions of the deck.

For these load cases, both the normal and tangential relative displacements and the stresses in the joints were calculated, as well as the principal stresses in the blocks and the global deformations of the bridge.

The response parameters of the structure were analysed and compared for all the phases of the analysis, starting with the results of the dead load and proceeding with the effects of the additional live loads.

Figs. 9a,b,c shows the results of the bridge linear elastic analysis in terms of deformed shape and the principal stresses due to dead load. Results of the non-linear analysis are not yet fully available since the experimental results were not completely processed by the time this paper was written. In a very near future, however, such results will be the object of a specific publication.

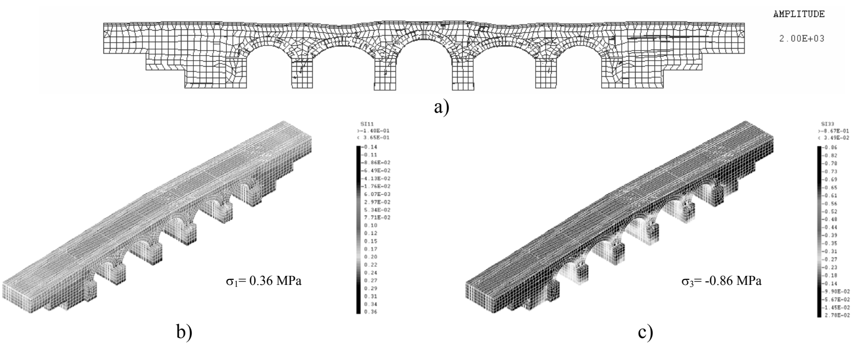


Figure 9 : (a) Deformed shape of the bridge due to dead load. (b) Tensile and (c) compressive principal stresses in solid elements (peak values in brackets).

Peak values of compressive and tensile stresses in the solid elements as well as the normal and tangential stresses and the joint element displacements due to dead load and to load case 2 are shown in Table 3. The later load case was found the most unfavourable, for which the percentage increments in the arches (relative to dead loads) are also included in the table.

Concerning these results, it is clear that peak tensions and compressions in the solid elements are compatible with the corresponding strengths as determined in the laboratory tests of stone blocks. Accordingly, the peak compressive stresses in joints between blocks have values which are quite common in this type of stone masonry structures, i.e. around or below 1 MPa and, therefore, far below the corresponding compressive strength.

Although quite significant percentage increments are found in Table 3, the traffic load influence in the bridge behaviour appears to be insignificant concerning the stress response. However the same conclusion can not be drawn for the deformations in the joints; the maximum increase (1850%) of the joint opening was found in the first longitudinal joint alignment in the arch intrados, right below the inner face of spandrel walls, and is in accordance with the peak increases (300%) of the slipping values (relative tangential displacement) in the same zone but in the transversal joints under the arch crown. Despite these significant increases of the results, the deformation values are still fairly insignificant and attest the great stiffness of this structure.

Table 3 : Stresses (a) in the blocks and (b) in the joints and (c) deformations in the joints.

a) Stresses in the blocks

Zone	Compression			Tension		
	DL MPa	DL+T MPa	Inc. %	DL MPa	DL+T MPa	Inc. %
Arches	-0.78	-1.32	+69	0,18	0,33	+83

b) Stresses in the joints

Zone	Normal stresses			Tangential stresses		
	DL MPa	DL+T MPa	Inc. %	DL MPa	DL+T MPa	Inc. %
Arches	-0.56	-0.94	+68	0.07	0.12	+71

c) Deformations in the joints

Zone	Opening			Closing			Slipping		
	DL (mm)	DL+T (mm)	Inc. (%)	DL (mm)	DL+T (mm)	Inc. (%)	DL (mm)	DL+T (mm)	Inc. (%)
Arches	0.02	0.39	+1850	-0.09	-0.15	+66	0.09	0.36	+300

DL – Dead load; DL+T – Dead load + Traffic load (standard 30 ton vehicle)

6 CONCLUSIONS

Throughout the previous chapters some details were presented concerning the design and construction of a new stone masonry arch bridge recently constructed. An overall description of the bridge and of its particular aspects was exposed as well as the design base criteria used for this construction. The different construction phases were also just briefly described, focusing on foundations, piers, abutments, arches, spandrel walls, backfill and secondary elements.

Also addressed were the materials used and the experimental tests (laboratory and "*in situ*" tests) carried out aiming at providing a good characterization of the materials adopted in the construction of the new Vila Fria Bridge. Besides the specific purpose for this particular case, this experimental campaign contributes to build up an extremely useful database to feed numerical models used in the bridge simulation with realistic parameters.

The basic conception of the bridge was not essentially conditioned for structural criteria and the design was first supported by classic pressure line methods. However, further detailed finite element structural analysis were also carried out, not only for additional support to design options but also for improving the knowledge about structural modelling of this type of structures by comparing the numerically obtained results with those achieved by the large scale monitoring activity of this bridge as addressed in a companion paper.

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