Structural analysis for the diagnosis of crackings in the Gothic masonry structures of the vaults of Serranos Bridge in Valencia, Spain

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ABSTRACT: Serranos bridge, built in 1402, the oldest bridge in Valencia for its Gothic ashlar masonry structure, is located in front of the old entrance with the same name in the walls of Valencia. In the preliminary research phase aimed at the Restoration of the Bridge, it is essential to know the historical-constructive evolution that provides the keys for the recognition of its current situation of deterioration, in order to accurately approach its restoration. The Bridge presents, as most noticeable structural pathology, cracks that run parallel to the arches in their joint with the vaults, mainly in the end spans of the bridge. In this phase of analysis described here, we aim at recognizing the structural behavior of the bridge and the current resistance conditions of masonry, by means of the theoretical-practical simulation, with the purpose of determining the coherence of the results and the conditions of the failure of material identified.

1 INTRODUCTION

In the area of consolidation of historical monuments, *structural stability* is a field with a high theoretical and practical interest that requires a specific evaluation and knowledge, closely related to architecture as discipline itself. It is very important then, to add to the architectonic restoration project that knowledge based on the relation between the historical evolution of the structural stability analysis and the current techniques related with the behavior of the masonry constructions, whether with stone or brick, and organize the development of the theory and critically analyze its validity within the framework of the modern *limit analysis* of masonry structures, that have been developed mainly by professor Jacques Heyman from the late 1960's. (Heyman J.1998).

The calculation of the thrust of arches and vaults has its first references in father Tosca's Treatise, (Tosca T.1794), one of the most cultured men of his time in Spain, and specially in volume 5 of his *Compendio* devoted to civil architecture in which he refers to these issues with enough detail to understand the context of statics at that time. His comments on the stability of masonry structures demonstrate a deep knowledge of their structural behaviour. In the case of the historic Serranos bridge of the city of Valencia, within the framework of the multidisciplinary team that has drafted the project of its complete restoration, starting from the graphic survey carried out in previous studies and based on a study from the constructive and geometric point of view, tests of materials, prospecting carried out in the cover, pilaster, backing, and land, as well as from the current state of the deteriorations observed in the masonry, we started to carry out the structural analysis of the bridge with the purpose of determining its stability, current masonry resistance conditions and with these data, be able to identify the causes of the damages detected and contribute to the most suitable solutions for its restoration.

Preliminary studies have revealed several damages in the stone, and as most noticeable structural pathology, crackings that run parallel to the arches in their joint with the vaults, mainly in the end bridge bays. With the purpose of carrying out a diagnosis of those structural damages, two lines of work have been followed.

Figure 1. Damage in the stone in the end bridge bays.

On one hand, surveys and geotechnical studies of the foundations of the bridge.

Figure 2. Foundations of the bridge.

These results have revealed that those damages are not due to foundation problems, and at the same time a structural study of the bridges by means of a numerical model to study their behavior faced with different load hypotheses which can explain the more relevant cracking conditions of the bridge and plan the proposal of structural restoration. The description of the numerical procedure carried out is presented below.

2 CALCULATION MODEL OF THE STRUCTURE OF THE BRIDGES

The development of the calculation model has been carried out through the program EFCID, a finite element analysis program developed in the MMCYTE Department of the UPV that uses CAD environments for the definition of this calculation model from the made accurate laser scanner survey.

2.1 *Analysis conducted*

The following types of analysis have been conducted: (López J. 1998) (Luccioni B. 1996)

a- Static linear

b- Dynamic with a modal-spectral calculation for seismic actions

c- Nonlinear Static with a damage isotropic model to characterize the breaking. (Lourenço P.B. 1996)

The objective is to determine the degree of safety against the actions planned on the bridge according to the current IAP standard, and to be able to determine the type of action that has produced the cracks in the lower part of the bridge vaults and parallel to the line of parapets.

The mortising on the bridge cover of installation conduits has been carried out.

2.2 *Definition of the mesh of the calculation model.*

The generation of the finite elements mesh is carried out by the graphical procedures of the design environment that the calculation program used allows, on the graphical medium of CAD drawing. This causes the resulting meshing to adjust to the contour of the form of the bridges with high accuracy, since their survey has been made by scanner-laser.

Superficial and volumetric elements have been used in order to make up the mesh. The solid elements model the mass parts of the bridges: piers, vaults, lateral arches and backing. The superficial elements have been arranged to model the asphalt layer of the cover and they are also used to support the traffic load. The layer elements connect with the solids through the mesh of nodes in the upper plane of the covers.

The characteristics of the flat elements used are:

Triangular isoparametric elements of three nodes and quadrilateral ones of four:

The superficial elements have two work levels whose effects operate in a disconnected form. Membrane effect with deformations and requests in the plane of its xy surface, and plate effect with flexions in the perpendicular direction based on the local z-axis.

2.3 *Membrane elements*

C. Felippa optimized triangle consists of three degrees of freedom per node, two transfers and one rotation. It is characterized because the efforts and deformations operate in their plane and are stresses $(\sigma_X, \sigma_Y, \tau_{XY})$ and transfers d_x and d_y and round z. referred to its local axes.

2.4 *Plate elements*

Triangular superficial flexion element, with three
knots, with three degrees of freedom per node (two rounds with respect to x-y and one transfer with respect to z), is the so-called DKT, discrete Kirchhoff triangle, based on Reissner-Mindlin plate model. Its characteristic efforts are bending moments M_x, M_y, M_{xy} and sharp T_x and T_y according to the local axes.

2.5 *Lamina elements*

Triangular superficial element of three nodes with six degrees of freedom per node. It is formed by the union of membrane elements. The quadrilateral elements are formed by the double partition by their diagonals in triangular elements.

Four parts corresponding to different constructive and mechanical characteristics have been distinguished in the mesh.

1. Ashlar masonry with unions in which the loss and degradation of mortar leads them to be virtually joined to bone. They are rot in their outside facade, which corresponds to the vaults of the bridge bays, the lateral arches, parapets and side spandrel walls, as well as the outside surface of the piers of the walls of the bridge. It has been modeled with hexahedral elements.
2. Backing of the vaults and piers, made up of coffer-work, with a very resistant concrete and in very good state. Formalized with solid elements.
3. A layer of hexahedral solid elements that form the pavement that tops the backing.
4. The fine asphalt layer placed on the stone pavement has been modeled with quadrilateral superficial layer elements, on which the traffic loads are entered.

The mortising produced has been materialized on the cover of the bridge and next to the parapets due to the conduit for installations. The triangular and tetrahedral elements are used only in those zones of transition between meshes of different discretization.

The model of the Serranos bridge consists of:

Hexahedral solid elements of 8 nodes.	25,978
Plate elements of four nodes.	3,300
Number of Nodes	35,794
Constrained nodes	1,340
Degrees of freedom.	115,944

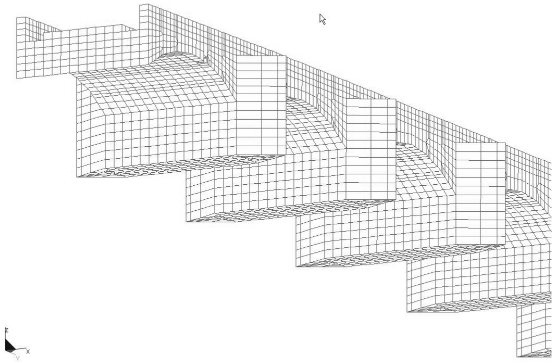

Figure 3. Model of the Serranos Bridge.

2.6 Characteristics of the materials

In previous studies test cylinders extracted from the bridge have been analyzed, corresponding to ashlar masonry structures. From the compression tests it has been obtained the load factor and the deformation module that is taken as parameter in the numerical model for the structural analysis. The values have been measured in two points of the ascending straight part of the chart of the simple compression tests. The results for the test cylinders of the Serranos Bridge are the following :

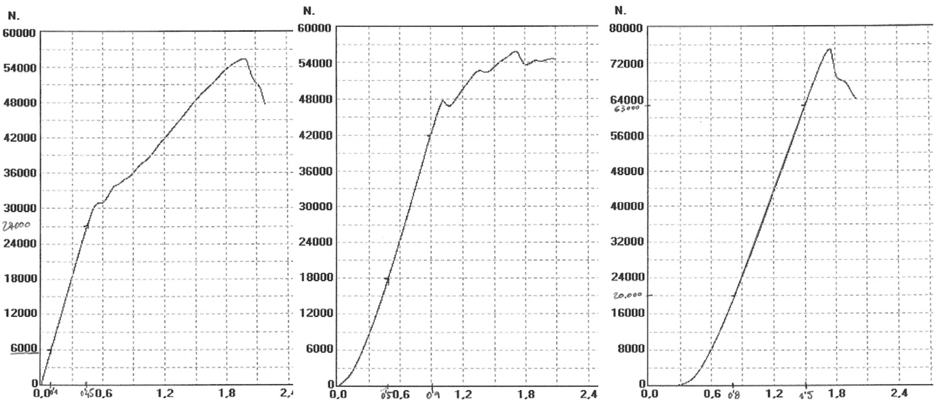

Figure 4. Stress-strain response for test cylinder .

Table 1

Test cylinders	Density T/m^3	C. Breakage N/mm^2	$\Delta\sigma$ N/mm^2	$\Delta\epsilon$	Module E N/mm^2
1	1.734	5.97	3.5225	0.00149	2360.085
2	1.760	6.14	3.1639	0.00200	1581.956
3	1.746	5.15	2.4608	0.00174	1413.214
Average	1.746	5.75			1785.085

The analysis of the results and the comparison with the values of test cylinders tested of the source quarry show that the ashlar masonry has undergone serious deterioration, increasing its porosity and consequently diminishing its density, its resistance and the deformation module. The values of the parameters of calculation for the backing material of the bridges have been obtained by comparison with data from the literature, since no test has been performed. (León J. 2001) (P.I.E.T 1971) (Eurocódigo 6 1997)

Properties of the materials considered in the numerical calculation model:

Coffer work backing material.

Density	2.3 T/m3.
Module of deformation.	9,000 N/mm2
Poisson's ratio	0.2
Compressive force	15 N/mm2
Tensile strength	0.5 N/mm2

Material of the stone pavement layer.

Density	2.4 T/m3.
Module of deformation.	11,000 N/mm2
Poisson's ratio	0.2
Compressive force	20 N/mm2
Tensile strength	0.6 N/mm2

Material of the asphalt layer.

Density	2.3 T/m3.
Module of deformation.	100 N/mm2
Poisson's ratio	0.2

Ashlar masonry. Serranos Bridge.

Density	1.918 T/m3.
Module of deformation.	1,425 N/mm2
Poisson's ratio	0.2
Compressive force	7.90 N/mm2
Tensile strength	0.15 N/mm2

2.7 *Evaluation of loads. Hypotheses considered*

Although this is a urban historic bridge, the estimation of the calculation actions on the bridge that have been entered in the numerical model to evaluate their tense-deformational conditions have been taken in compliance with the Regulations on the actions to consider in the project of road bridges, IAP of the Department of Public Works. This regulation specifies the actions and safety factors for the construction of new bridge. The Serranos bridge was built centuries ago and throughout its history it has been under different types of loads and values, it is impossible to reproduce this history of loads. During the last century, it has been put progressively under traffic loads for which it was not intended.

The objective of the structural analysis performed is to determine with a degree reasonable of accuracy the conditions of the bridge under the actions undergone and to attempt to determine the causes of the structural pathologies that have appeared at present time. For that reason, the regulations in the IAP have been taken as reference.

The load hypotheses considered are the following:

Hypothesis 1. H1 Weight of the bridge. Determined according to the densities of the different materials that compose the bridges, which have been defined in the section on materials characteristics.

Hypothesis 2. H2 Overload of 10 KN/m2 to be considered in roads with trucks according to the reference of NBE-88.

Hypothesis 3. H3 Overload of 4.0 KN/m2 of vertical action all over the cover (IAP art.3.2.3.1.1.a1) . It corresponds to the overload use as bridge for pedestrian use exclusively, or simultaneously used to the load of vehicles.

Hypothesis 4. H4 Loads of vehicles of 600 KN with parallel longitudinal axis to the road, divided into six loads of 100 KN according to IAP regulation art. 3.2.3.1.1.a2. In this hypothesis the higher number of vehicles will be placed in a row, one after another throughout all the bridge.

Hypothesis 5. H5 the same load than H4, but only vehicles are placed in the center of the space of each bridge bay.

Hypothesis 6. H6 is applied to the load of the H4 hypothesis, placing the vehicles on each one of the supports

Hypothesis 7. H7 Applied to the alternation of loads by applying the loads of vehicles in the centers of the even spaces of the complete the bridge.

Hypothesis 8. H8 horizontal load of finishing and springing (IAP art. 3.2.3.1.1.b1.) It is applied in the direction of the axis of the platform of the board and operates at the level of the surface of the pavement as a uniformly distributed action.

Hypothesis 9. H9 inclined load of centrifugal action of the vehicles in the entrance and exit of each bridge. It is applied in the center of the initial and final span of the bridge. The load is evaluated according to IAP art.3.2.3.1.1.b2.

The centrifugal force Fc applied is:

$$F_c = K \cdot M \frac{V_e^2}{R} \tag{1}$$

where M = mass of the overload 60,000 Kg; V_e = speed in the section 14 m/s. corresponding to 50 Km/h; R = radius in plant 15m; K = factor of adimensional distance.

$$\frac{231}{V_e^2 + 231} = \frac{231}{196 + 231} = 0.54$$

$$F_c = K \cdot M \frac{V_e^2}{R} = 0.54 \cdot 60{,}000 \frac{196}{15} = 423{,}360N$$

The vertical load is reduced to

$$0.54 \cdot 600 = 324KN$$

Hypothesis 10. H10 Seismic loading

The seismic loading have been considered in accordance with Art.1.2.3 of the NCSE-02 standard and instruction (IAP art. 3.2.4.2.). A basic acceleration of ab= 0.06 g. is taken, specified for the city of Valencia. The construction is considered of Special Importance (Art. 1.2.2 of NCSE-02). and (IAP art. 3.2.4.2.1.). The calculation is performed by means of the analysis of response spectra. A three-dimensional model of the structure with 6 degrees of freedom per node is carried out, without restrictions nor simplifications.(Code UIC 1995) (ACI 1999) (Hendry A. 1998).

Given the special case of historical bridge and the high value of its "useful life" a coefficient of higher risk is taken by considering it as 1.5.

The combinations carried out are:

C1: 1H01 + 1H02
C2: 1H01 + 1H03 + 1H04
C3: 1H01 + 1H03 + 1HO5
C4: 1H01 + 1H03 + 1HO8
C5: 1H01 + 1H03 + 1HO9
C6: 1H01 + 1H03 (Combination for bridge without road traffic)
C7: 1H01 + 0.5H3 +1H10

With combination 5, considered as the most unfavorable for the purpose of studying the cracking produced, the nonlinear analysis of damages has been performed. (Hanganu A.D.1997)

2.8 *Analysis of results*

The analytical study of the bridge, through the program of calculation by finite elements EFCID, from the calculation models, load hypotheses and
combinations, and materials described above, has shown the following issues as the most outstanding features of the structural behavior:

The results of the behavior of the bridge, in its analysis with static gravitational loads, in any of their combinations, as well as earthquake dynamics, the latter taking into consideration a period of recurrence of 500 years, are acceptable values that do not imply possible structural deteriorations.

In the linear analysis, the values of the compression membrane stresses, in any of the directions, are not relevant with any of the combinations of static load analyzed, with quite moderate values of the compression stresses in all of them, mainly below 0.3 N/mm2, very distant from those allowed for the ashlar masonry tested.

The tensile stresses, under these same combinations of static load, are nonexistent or negligible, which does not justify any of the existing damages in the bridge.

The hypothesis of seismic loading, combined with gravitational statics, does not produce significant increases of the tensional state. From the nonlinear analysis with models of damages, for values of main stresses in ashlars that exceed 6N/mm2 in compression or 0.1 N/mm2 in tensile strength, and values in coffer work backing of 15 N/mm2 in compression and 0.5 N/mm2 in tensile strength, shows the absence of significant damages in any of the combinations of static gravitational load analyzed as well as in the combination of gravitational loads plus seismic loading, therefore it can be concluded that the behavior under these actions is safe.

Regarding the traffic actions considered in hypotheses 8 and 9, of braking and centrifuge action in circular layouts respectively, combined with gravitational statics, described in Combinations 4 and 5, the tensions of traction in the inside face of the first vaults of the bridge, increase considerably, reaching values of up to 0.5 N/mm2, which justifies cracking conditions that correspond with those appeared in the vaults of the bridge, mainly in the end spans.

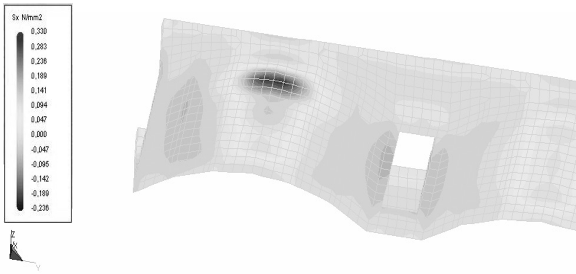

Figure 5. Axial stresses X direction.

These stress conditions values are coherent with the nonlinear analysis with damage models, (12) for such values of main stresses that in the previous combinations, that is, 6N/mm2 in compression in ashlar or 0.1 N/mm2 in tensile strength, and values in coffer work backing of 15 N/mm2 in compression and 0.5 N/mm2 in tensile strength, which reveals the appearance of significant damages of around 0.8 in ahslar masonry of the bridge, within a scale that ranges between 0, absence of damages and 1 collapse of the material.

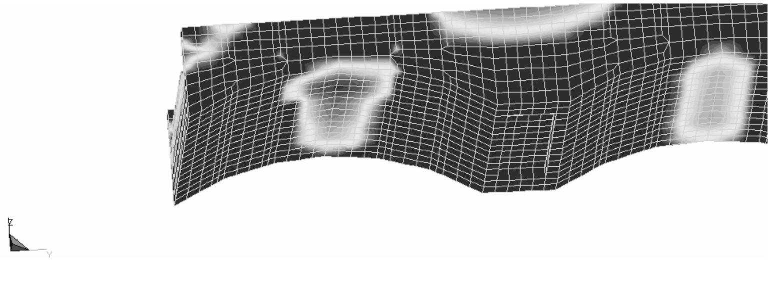

Figure 6. Model of Damages

These stress conditions values, may be due to the way in which traffic affects the bridge, and that those horizontal stresses are transmitted to the joints of the ashlar masonry structure causing the most significant cracking of the vaults in its inside face.

From the above it can be concluded that, road traffic is the cause of the cracking in the vaults. The cracks that appear parallel to the arches in their union with the vaults, in the end spans of the bridges, are due to the horizontal actions caused by the road traffic in the curved layouts at the entrance and exit of the bridge.

With respect to the calculation of the anchors that can be arranged for the bonding of the existing cracks produced by the tensile stress in the masonry, its approximate value is the following:

The maximum tension produced in the zones of the crackings, for a situation of light traffic, is 0.10 N/mm2 of traction in the inside surface of the vault, which is already a compression of 0.05 N/mm2 at a distance of 0.5 m. over that surface, therefore the anchors are appropriate as repair system approximately arranged 20 cm. from the inside face of the voussoirs of the arches.

The mechanical capacity of the anchors in the tensile area, for an arc-length of 1m. will be:

$$F = 0.10 N/mm^2 \cdot \frac{340}{2} mm = 17T/mm(1.7T/m)$$

Applying a safety factor of 3

$$Ftotal = 5.1T/m$$

This force has to be distributed between the number of anchors to be arranged in that surface based on the mechanical characteristics of the material selected.

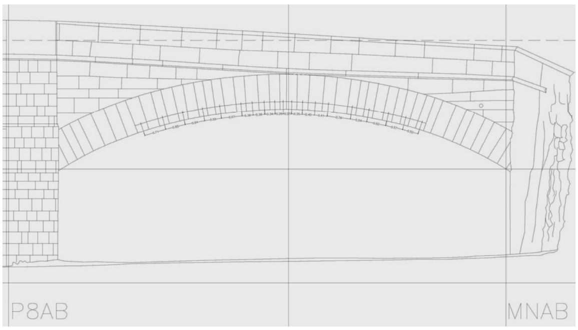

Figure 7. Details of anchors.

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