Seismic vulnerability evaluation of Devil Bridge on Sele River

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ABSTRACT: Object of the present study is the masonry bridge on Sele River, near Salerno, built in 1871 by Eng. Fiocca, still standing today. This bridge has a single semi-elliptic arch, spanning 55.00 m, with a rise of 13.55 m and a width of 5.40 m; in order to reduce the dead loads the inner tympani were lightened employing ring-like vaults. The first step of this research has been the study of the bridge static behaviour, deduced by the material survey of the existing structure, but also analyzing, verifying and enriching matters treated in the Report of Eng. Sacco, a Fiocca's collaborator, in which all phases of Devil Bridge designing and building are fully described. The second step has been the evaluation of its seismic vulnerability: a three-dimensional finite element model has been implemented in ABAQUS code. To assess the bridge seismic capacity, non-linear pushover analyses have been carried out.

1 INTRODUCTION

Masonry arch bridges have always had a particular role in the framework of civil infrastructures. So even in the second half of the eighteenth century and first half of the twentieth century, when the largest part of the modern railway and road system was already being build employing iron, concrete and suspension bridges, masonry arch bridges were still used.

The masonry arch bridges designed by Eng. Giustino Fiocca (Castel di Sangro, 1821 – Napoli, 1877) are quite interesting as they represent the two last examples of masonry arch bridges dating to this period: the Hannibal Bridge on Volturno River, destroyed during the Second World War (Fig.1), and Devil Bridge, on Sele River still standing (Fig.2).

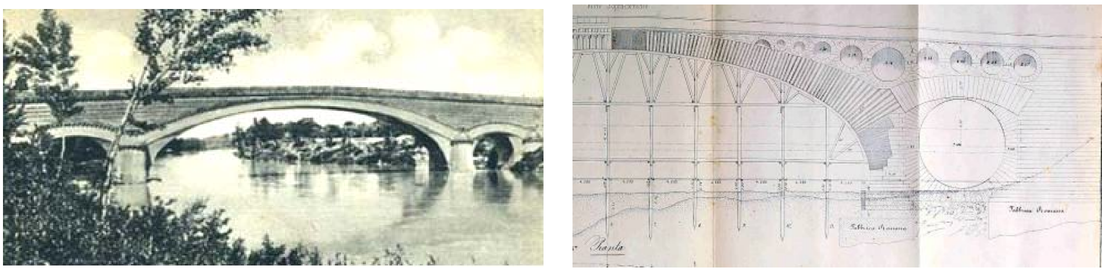


Figure 1 : Hannibal Bridge on Volturno River, view (Séjourné, 1913) and cross section (Sasso, 1872)

This last one is the object of the present study, where its structural behaviour is analysed.

Eng. Fiocca, whose quotations on several treatises – even Séjourné published some of his projects – testify his international notoriety, was trained at the School of Application of Bridges and Roads of Naples, in which he was admited in 1841, and so represents a member of the new class of Engeeners who worked in the South of Italy in eitheenth century and whose technical knowledge was quite advanced, formed on the same model of the French School of Bridge and Roads. In designing and building the Devil Bridge, and also the Hannibal one, he was coadiuvated by Eng. Sasso, who has described all the phases of those realizations in detailed Reports, written in 1871 and 1873, now kept in the Neapolitan National Library.

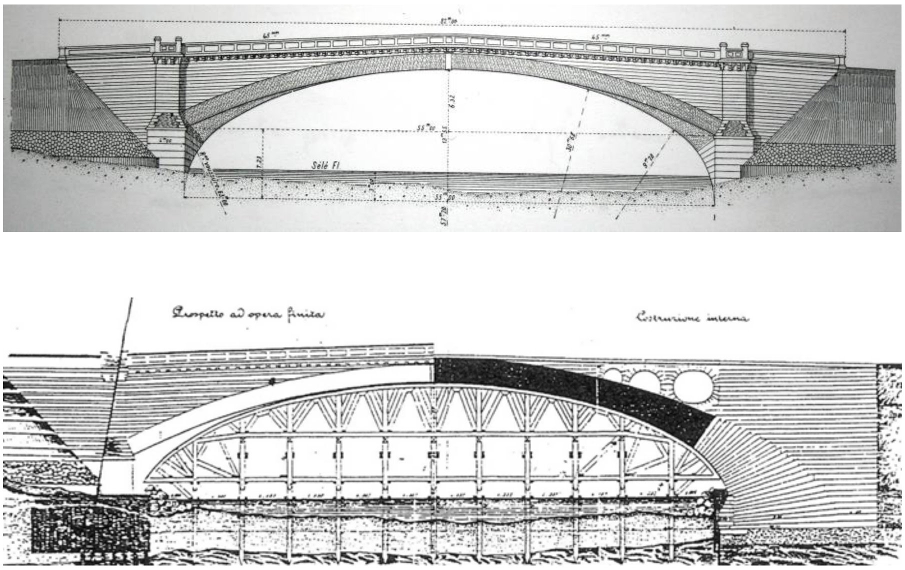


Figure 2 : Devil Bridge on Volturno River, view (Séjourné, 1913) and cross section (Sasso, 1872)

So, the first step in the analysis of Devil Bridge stability, together with the material survey of the existing structure, has been a deep study of the Report of Eng. Sasso, comparing the different matters described there with background knowledge to which Sasso makes reference as: empiric formulas for dimensioning arches, employed materials behaviour and graphic methodologies for static evaluation. Next, the evaluation of the bridge seismic vulnerability, taking into account the recent Seismic Technical Code, has been performed. At this aim, a three dimensional finite element model has been implemented in ABAQUS code. The geometric model of the structure has been partitioned in different regions in order to take account of the different behaviour of the structural masonry and of the backfill. Solid elements have been used and the concrete plasticity model has been chosen for the constitutive law. To assess the bridge seismic capacity non-linear pushover analyses have been carried out.

2 FIOCCA'S DEVIL BRIDGE

2.1 Historical survey

The bridge studied here, located near Salerno on Sele River, was built on the ruins of a Roman bridge, and was finished in 1872. In 1844, Alexander Cottin began the construction of a suspension bridge "with masonry abutments and iron chains" which, due to several difficulties, was not completed. Between 1864 and 1866, Eng. Francesco Giordano built an iron bridge on stone piles, but it collapsed once completed. After this event, the Salerno Province Deputation announced a competition and eight projects were presented; Fiocca proposed a masonry arch bridge, with two spans, but the Deputation chose that with a single steel girder on stone abutments. After few days, the Hannibal Bridge, designed by Fiocca, was opened on the Volturno River and the Deputation was so positively impressed, that ordered to Eng. Fiocca the design of the new bridge on Sele River, with the same characteristics of the Hannibal Bridge. The bridge building, on Fiocca's plan, began in 1871, adopting the construction technique already tested in the previous bridge on Volturno River, and its total cost turned out of 340,000 £.

The studied bridge can be ascribed to the typology of arch masonry bridges with lightened tympanum, developed in nineteenth century. Two different tympanum lightening typologies can be envisaged basing upon the disposition of the lighting vaults: longitudinal lightening, made with barrel vaults parallel to the front planes of the bridge; transversal lightening, with the barrel vaults disposed perpendicular to the main arch. In the first building system, the vaults stay on masonry walls leaning upon the main vault extrados; so the front walls of the bridge are bearing structures. In the second disposition, lighting vaults are based on masonry walls perpendicular to the front walls, which consequently result unloaded and become simple closing walls, so that large openings can be cut, and aeration of the masonry is obtained. The Devil Bridge presents the second one, as can be deduced by the following description.

In Campania Region another example of masonry arch bridge with lightened tympanum can be found, built at the beginning of nineteenth century: the arch bridge of Niccolini in Villa Floridiana in Naples (Fig.3). As pointed out in Ceraldi et al., Niccolini can be considered a forerunner in the employment of this building technique, which involves many advantages as reduction of the thickness of the front walls, and consequently of the load upon the whole structure, also on the centrings during the vault making.

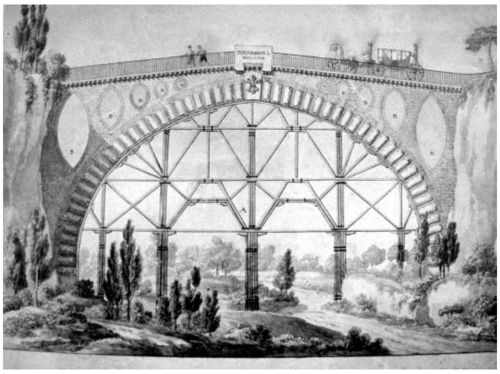


Figure 3 : Niccolini's own hands drawing, St Martin Museum

The deep analysis of the intrados shape of the built arch has shown that for its layout, Fiocca employed a multi-centers curve - of five centers - named anses de panier arch. This procedure for tracing an arch profile was already used by Perronet in his well-known Neuilly Bridge.

The vault, made with five parallel arches, was built employing three superimposed rolls, with directrix parallel to the intrados curve, at the aim of avoiding large concentrations of loads on the centrings, which were dimensioned to support only the load of the first roll. The three layers were forced together by the insertion of little wedges, properly envisaged for this task. After the building of the first layer, the central point of the timber centrings didn't showed any vertical displacement, which was observed later when the second layer was finished, measuring 0.02 m, remaining the same until the ending of the vault building. In any case, to compensate vertical displacements which could appear in the construction meantime or after removing centrings, the timber structure was built with an over height in the key position of 0.13 m. About the removing of the centrings, Sacco, in his Report, wrote a quite interesting note: starting from the observation that there isn't any horizontal thrust until the vault stands on the centrings, and then in this situation the thrust line is only an ideal geometric construction, while that thrust attains its maximum value when removing the centrings, he deduced that if this removing is too fast, the thrust could reach a value larger of this expected maximum value. Consequently, the timber structures were dismantled very slowly, measuring the key displacements in the meantime, whose largest value was of 0.34 m. The author compared this value with those of other existing masonry bridges, quoting that the arch of the Neuilly Bridge, which had a span of 39.00 m and a maximum rise of 9.75 m, showed a key vertical displacement of 0.32 m.

2.2 Sizes and materials

The in situ survey allowed drawing completely the bridge, which has a single roadway, 82.00 m long, 7.00 m large, Fig.4a. The structure is constituted of a single arch, with a span of 55.00 m, 5.40 m large, keystone height of 2.00 m, maximum rise of 13.55 m, and having, near to the impost, a splay of 0.80 m, Fig.4b. The splay frontal semi-circular arches have a radius equal to 63.00 m and maximum rise of 6.32 m. In the opinion of the authors, this splay hasn't only an aesthetic effect, but meet also strength and stability requirements.

The part of the vault near the impost, is made of tuff, completely covered by plaster with bush hammered ashlars work. The remaining part is made of bricks, Fig.5a.

(a) (b)

Figure 4 : (a) Frontal view of Devil Bridge, (b) The vault splay

(a) (b)

Figure 5 : (a) Detail of the impost, (b) Detail of the abutment

The employment of two different materials, tuff and bricks, to built the vault is obviously due to their compressive strength and their density, so the lighter material is used to lighten the upper part of the vault, until the rotation point, were crushing can appear. Abutments are made of calcareous stones (Fig.5b) inclined following the arch profile, offering a larger bearing capacity where larger thrusts are expected. To reduce the load on the large depressed vault, the tympanum was lightened with ring-like vaults, made of tuff ashlars, hidden by the closing front walls, (Fig. 2b). The whole building is covered by plaster and enriched by a bricks cornice; the keystone is made of Vesuvius stone.

Eng. Sacco wrote that great attention was devoted in choosing building materials: bricks for masonry were brought from Gaeta, because of better quality than those available in situ; mortars were made with pozzolana coming from Bacoli and from Vesuvius, mixed together in different proportions, while hydrous lime was imported from Marseille. Tests and chemical analyses were made to ensure that the mortars had carbonation times speed and different for each roll.

The visual inspection allows to say that the bridge is in a quite good state, only showing some surface finishing decay.

3 THE STRUCTURAL BEHAVIOUR OF THE BRIDGE

3.1 Empirical rules for building masonry arch bridges

A long time before the theories of statics and strength of materials were employed for dimensioning the structures, some empirical rules had been used for designing frequently recurring building elements such as pillars, vaults, foundation. Until the nineteenth century, these empirical rules, condensed into concise geometrical terms, were used, together with other proportionality rules chiefly designed for aesthetic purposes. Due to their empirical nature, they did not take account of essential parameters influencing the construction stability, like the materials strength, the arch weight and lives loads, as they were essentially based on the observation of traditional constructions. Making use of these rules allowed the technicians to provide an answer to the most frequently questions arising during construction and to establish the validity of their formulations.

(1) The first question posed in the design of a masonry arch bridge was to fix the intrados curve profile

Until the eighteenth century, the choice of the intrados curve was limited to few recurring types as the circular and segmental arch. One of the first rules was not using circular arches with a ratio (s) rise (f) to span (L) less than s = 1/9 or greater than s = 1/6. At the end of the century, it became recurrent employing “anse de panier arches”, developed by laying out the intrados curve by means of a multi-centers curve, of three, five, seven and even more centers. In literature, different values related to the choice of the ideal ratio rise/span are quoted, especially applied to “anse de panier arches”. In particular, with s = 1/3 and L < 10 m, the employment of a three centers arch was advised; in the case in which 10 < L < 40 m, with s = 1/3, a five centers arch was preferable. Instead, for L > 40 m and s = 1/4, a five, seven or nine centers arch was recommended.

In the case of Devil Bridge, being the rise and span equal to 13.55 m and 55.00 m, respectively, the ratio is s=1/4, so a five centers arch has been employed.

(2) Thickness of keystone

In dimensioning keystone thickness, bridge builders used empirical rules giving it as a function of the following parameters: the span “L”, the rise “f” and the materials used. The architecture treatise writers, as Alberti, Palladio and Serlio, established that the optimum ratio between the crown joint thickness (e) and the span should be equal to 1/15 (Alberti), 1/12 (Palladio) and 1/17 (Serlio).

In the case of a big arch, the ratio e/L ought to be equal to 1/24; so, being the span of Devil Bridge 55.00 m, its keystone thickness “e” ought to be 2.29 m.

Other empirical rules for arch sizing, widespread in contemporary literature, gave evaluation of the thickness “e” as a function of the span “L” and the curvature radius “r”, Sasso:

Perronet assumed e = k1+k2L with k1=0.325 and k2 = 0.035; in this case e = 2.25 m.

Gauthey assumed e = k1+k2L with k1=0.67 and k2 = 0.035; in this case e = 2.59 m.

Déjardin assumed e = k1+k2L with k1 = 0.30 and k2 = 0.05r; in this case r = 36.4 m and e = 2.12 m.

Léveille assumed e = k1+k2L1/2 with k1=0.33 and k2 = 0.33; in this case e = 2.77 m.

Dupuit assumed e=kL1/2 with k =0.20; in this case e = 1.49 m.

The keystone thickness of Devil Bridge is equal to 2.00 m, for a span of 55.00 m, a value which isn’t in agreement with the above relations. The choice of Eng. Fiocca was based on the analysis of existing bridges and on his previous experience in building the Hannibal Bridge.

(3) Arch thickness at the springers

The arch thickness “e’” at the springers was calculated as function of keystone thickness “e”, the angle φ which the normal at the intrados in any point forms with the vertical axis and a parameter “k”. The values of k depend upon the intrados arch profile and the ratio rise/span of the arch.

Déjardin assumed e’ = e/cosφ;

Croizette-Desnoyers assumed e’ = ke.

The arch thickness at the springers of Devil Bridge is equal to 2.80 m, for a keystone thickness of 2.00 m, and φ=60°, perfectly according to the value assumed by Déjardin. Applying the Croizetté-Desnoyers formula, for k=1/4, the value obtained is equal to 3.20 m.

(4) Abutments and piers thickness

Another important aspect quoted in treatises on masonry arch bridges was the dimensioning of the abutments and the piers. The more common relations give the pier thickness "S" as a function of arch keystone thickness "e".

Perronet suggested the following relation: S = 2e. The piers thickness of Devil Bridge is equal to 4.00 m, for a keystone thickness of 2.00 m, perfectly according to Perronet's assumption.

3.2 Design by Méry's method

To verify and dimension the arch of the Devil Bridge, Fiocca employed the well-known graphic procedure of Méry based on the principle of " minimum thrust".

At the aim of determining the thrust line, the vault was divided in twelve parts: the first ten were 2.47 m large, while the second last was 2.80 m and the last one 2.00 m.

In Fig.6, the thrust line, so as traced by Fiocca, is reported. According to the assumptions of Mery's theory, the thrust line was constructed imposing that, at the keystone, the thrust goes through the third upper mean, and at the impost, trough the third bottom mean. This disposition corresponds to the most unfavorable assumption.

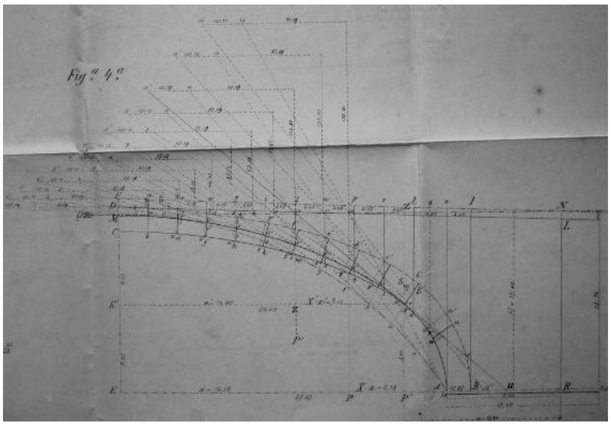


Figure 6 : Devil Bridge thrust line by Fiocca

The evaluation of the horizontal thrust value, equal to 2119.25 kN, gave a maximum compressive stress for the keystone equal to 1.06 MPa, less than the compressive strength of Gaeta bricks, determined by experimental compression tests and equal to 8.96 MPa. So the occurrence of crushing in the keystone was avoided, as it was over-dimensioned.

Once a thrust line equilibrating the applied loads had been founded inside the thickness of the arch, Fiocca faced another problem: the arch drift on the abutment ought to be inside its thickness, and at a distance from the outer sides such that live loads couldn't change the situation. In this way he came to defining a stability parameter "m" avoiding the overturning of the piers. Building practice fixed a range for m: 1.5 < m < 2. For Devil Bridge m is equal to 2.36, so also in the case of abutments there seems to be an over-dimensioning.

In his structural scheme, Fiocca didn't considered the enlargement of the vault due to the splay from the key towards the impost. So he gave not a structural role to the splay. The authors, always using the Mery's method, have verified its structural function: the gradual enlargement of the vault was necessary to avoid that during the removing of the centrings the thrust line could move towards the extrados, determining a rotational section.

Another study the authors have made concerns the interpretation of the tympani lightening system. They have verified that the thrust line is quite the same with or without the presence of the openings in the backfill. So the designer choice of this typology of bridge had to be due to other reasons, even the will of avoiding over-loading of the centrings and consequent too large displacements.

The originality of Fiocca's approach isn't in the graphic method employed, but in the procedure he follows to trace the thrust line. He fixes as initial points those corresponding to Mery's hypothesis; then he modifies the thrust line until he finds the point of maximum thrust, which will be the point where a rotation of the masonry block can happen, or, in modern terms, the point corresponding to a plastic hinge. So Fiocca, trying to find a thrust line inside the arch profile instead of passing through the characteristic points of Mery's method, intuitively anticipates structural consideration which will be basilar in the theory of Heyman.

3.3 Assessment of seismic behaviour by finite element analysis

To assess the seismic behaviour of Devil Bridge, a nonlinear static (pushover) analysis has been performed.

A three dimensional model of the masonry arch has been developed in ABAQUS for numerical investigation. The three dimensional solid model of the bridge has been implemented in a computer aided design system and then has been imported into the finite element code to generate the geometry of FE model.

The geometric model has been partitioned in different parts in order to take into account the different behaviour of structural masonry and fill material.

The solid parts of the model have been meshed with the C3D4 4-node linear tetrahedron element, with three degrees of freedom at each node, namely the translations along the nodal x, y, and z directions. The average dimension of mesh size is 1.50 m.

The concrete damaged plasticity model has been selected for constitutive law. The model is based on isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behaviour of brittle material. The yield function of Lubliner is considered as failure surface, with the modifications proposed by Lee and Fenves to account for different evolution of strength under tension and compression (ABAQUS, 2007).

The values of parameters considered in the analysis are reported in Table 1. Two material models have been considered for structural masonry and fill materials. The elastic and plastic parameters are referred to the values that have been considered in the original design, which were confirmed form experimental investigation. As far as compression behaviour, an elastic-perfectly plastic response has been assumed. The post-failure behaviour of cracked masonry in tension is modelled with linear strain-softening, defined by the fracture energy GFI. The considered value of dilation angle Ψ characterizes the non-associated potential plastic flow.

Table1 : Materials properties employed in the numerical analysis

Model	Material properties						
	γ (kN/m^3)	E (MP)	ν	Ψ (Deg)	σ_c (MPa)	σ_t (MPa)	GFI (N/mm)
Masonry	18.0	5000	0.2	20	2.0	0.1	0.1
Fill material	16.0	1000	0.3	20	0.5	0.02	0.1

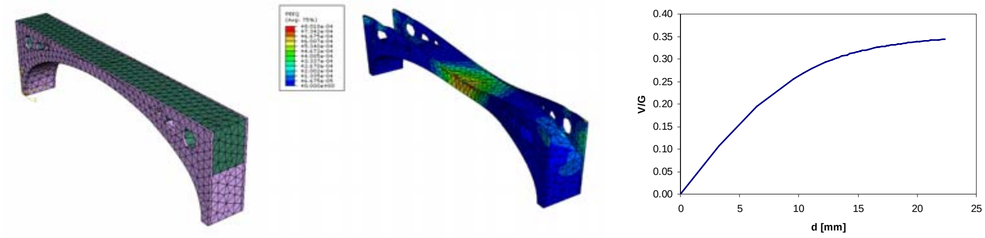
With regard to the load modelling in pushover analyses, two steps have been considered. In the first step gravity loads have been applied to the FE model and in the second one a uniform distribution of accelerations along the horizontal direction has been considered.

As far as the boundary conditions are concerned, full restraints have been assumed at the piers of the structure in the performed analyses.

In Fig.7a-c, the results of the analysis in terms of distribution of damage and pushover curve are shown.

The analysis shows that the lateral capacity of the bridge is attained for compression failure at the key of the arch.

The pushover curves are expressed in terms of base shear to weight of the bridge ratio and relative displacement of the control point. The results show that the capacity is attained for a value of the V/G ratio of about 0.35.



(a) (b) (c)

Figure 7 : (a) The finite element model; (b) distribution of equivalent plastic strain under lateral loads at collapse. Determination scale factor: 200. (c) Pushover curve.

4 CONCLUSIONS

The Devil Bridge is one of the latest examples of masonry arch bridges built at the end of eighteenth century. All phases of its design and realization are described in a Report of Eng, Sasso, which results in a theoretical and practical compendium showing the deep knowledge of structural problems and of practice rules envisaged to meet them.

In this work the arch bridge stability has been evaluated, basing upon a detailed survey, employed materials analysis and an accurate examination of the historical sources. Application of the empirical rules for dimensioning and verifying arch structures leads to satisfactory results and in many respects the structures seems over-dimensioned. The great experience of Devil Bridge author, Eng. Fiocca, let him adopting innovative solutions.

The seismic analysis shows that the lateral capacity of the bridge is attained for compression failure at the key of the arch.

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