Acta Polytechnica CTU Proceedings **33**:105–111, 2022
https://doi.org/10.14311/APP.2022.33.0105

RC DECK - STIFFENED ARCH EXISTING BRIDGES: SIMULATED DESIGN AND STRUCTURAL ANALYSIS

Giovanni Crisci[a,*], Francesca Ceroni[a], Gian Piero Lignola[b], Andrea Prota[b]

[a] *University of Naples Parthenope, Department of Engineering, Centro Direzionale C4, 80143 Naples, Italy*

[b] *University of Naples Federico II, Department of Structure for Engineering and Architecture, via Claudio 21, 80125 Napoli, Italy*

[*] corresponding author: giovanni.crisci@uniparthenope.it

Abstract.

The 20th century is known as the age that gave birth to the largest reinforced concrete structures. Many applications of this new material were realized at that time, both from a theoretical and practical point of view. With reference to bridges, the engineer Robert Maillart achieved a new concept of arched bridges, characterized by very stiff deck beams and slender and wide vaults, i.e., the "Deck-Stiffened Arch". The paper deals with the study of such bridge typology, particularly widespread in Italy around the 50s of the 20th century. While, nowadays, calculation tools allow developing very refined structural modelling, in the past very simple structural schemes were adopted in the design phase in order to simplify the calculation effort. The study starts from a "simulated design" of such a bridge typology adopting a reliable geometry and following the design rules and the simplified structural schemes of the time and, then, by means of a refined three-dimensional model, the performance of a typical "Maillart-Type Arch" bridge is analysed.

Keywords: Maillart-Type Arch bridges, reinforced concrete arch bridges, structural analysis.

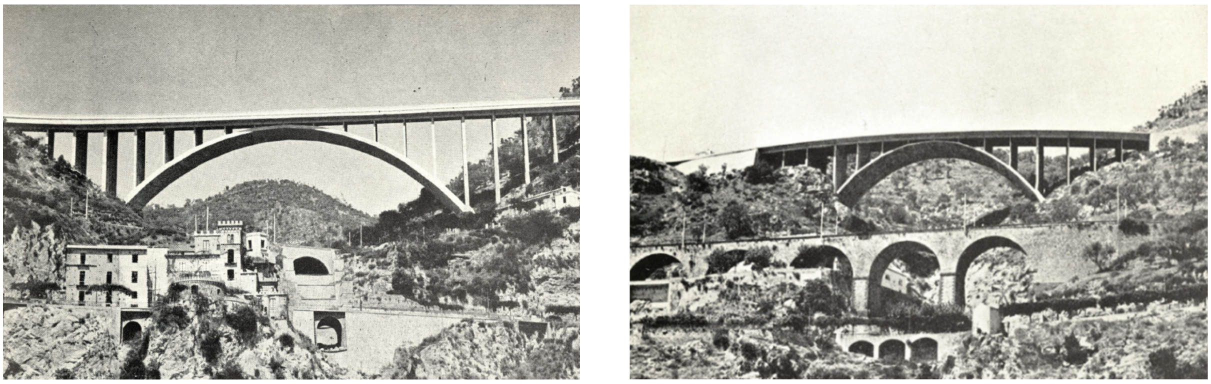
1. Introduction

The arch bridges are one of the oldest type of structures and are still today widely used to cross deep ravines in mountain areas. Reinforced Concrete (RC) arch bridges represent a suitable alternative to the classic "girder bridges", both on the aesthetical and functional points of view. The arch is, indeed, able to optimize the compressive strength of concrete or masonry materials by transferring the gravitational loads through a distribution of axial compressive forces. Due to the development of new building techniques and high - performance materials, the diffusion of RC arch bridges increased considerably.

RC arch bridges can have different forms such as "tied - arch bridges", "deck arch bridges" and "through-arch bridges". Generally, the "deck arch bridge" is the most widely preferred typology because the arch sustains the gravity loads by means of compressive stresses. The key components of an RC deck arch bridge are: the deck beam (i.e., the superstructure), the piers (i.e., the piers that connect the deck to the arch), and the arch. Within the "deck arch bridge" typology, the paper focuses the attention on the "Deck-Stiffened Arch Bridges", also known as "Maillart-Type Arch Bridges", aăparticularly widespread typology in Italy, built between the 40s and the 60s of 20th century.

In large - span bridges, the contribution of the deck beam to the bearing capacity of the whole structure is completely ignored, but it is assumed that the deck beams only bear and transfer gravitational and accidental loads over the arch. On the other hand, in medium - span bridges, the contribution of the deck beam, as long as it is continuous, is considerable and it would be uneconomical not to take its contribution into account.

The "Maillart-Type Arch" bridges (see Figure 1a and 1b) are an example of the so-called collaborative deck structures, in which the deck is not only a simple element for transferring the loads, but it contributes to the bearing capacity of the whole structure. For typical RC arch bridges, the inertia of the deck beam is very small in comparison with the inertia of the arch cross section and, thus, the arch bears most bending moments (and shear). The Swiss engineer Robert Maillart, expert of bridges and RC structures, at the beginning of 20th century, reversed this idea, since he conceived a new bridge typology, which took his name, made of a slender and wide vault, characterized by a low moment of inertia, and a very stiff deck beam. The two elements were connected by means of slender piers, often wall-like and lightened by central windows. The inversion of the classic ratio of arch and deck stiffness was a very important expedient, since it led to the significant reduction of the bending (and consequently shear) forces in the arch, which worked mainly in compression (i.e. a funicular arch). This has the advantage of simplifying the scaffoldings for the construction of the arch, which was usually built in deep ravines and ended with a structural scheme that is an inverted suspended bridge.



a) b)

Figure 1. Example of Maillart-Type Arch Bridge.

Several viaducts operating in Italy and inserted in the national highway or motorway network have now reached the so-called "service life". Surely these structures have been designed and built with a very different philosophy from the current design principles and with different design actions. Recent collapses, such as the one involving the Polcevera Viaduct in Italy, have highlighted the need of a good balance between social, economic, and environmental evaluations in order to attain a Sustainable Development. For bridges, sustainability can be interpreted as the resilience that the structure has to offer against the ever increasing environmental and loading conditions. Thus, for a suitable sustainable approach, it is necessary to understand the structural concept that governed the design of the bridges in order to catch their resilience over the time.

Within the RC deck arch bridge, in this paper, the attention is focussed on the "Maillart-Type Arch" typology. Firstly, the design methodology of the time for such a bridge typology was investigated, then a "simulated" design of a reliable geometry was carried out according to simplified models and assumptions in agreement with the methods of the time. The elements of the simulated bridge have been designed and verified according to the "Allowable Stress Method (ASM)" or "Working Stress Method (WSM)" [1] and under the design loads of the time. Lacking high computational capacity, the typically adopted design schemes at the time were easier than those currently adopted. Two-dimensional models have been, indeed, adopted for the design of structural elements and some simplifications have been assumed about their constraints and working conditions. In turn, the real geometry of the case study has been implemented in a Finite Element (FE) software and, under the loading conditions of the time, the structural elements have been verified again according to the WSM.

2. Design of structural elements for the case study

2.1. Definition of geometry, materials and loading conditions

A simple model of a "Maillart-Type Arch" bridge, i.e. a RC deck arch bridge, was assumed according to the scheme of Figure 2a as case study for the analysis presented in this paper.

The bridge is made of a slender vault with a cross section having constant dimensions of 6m × 0.2ăm. The shape of the arch was designed in order to be funicular of the gravitational loads corresponding to the self-weight of the entire structure. This means that it is subjected to axial compression only, without any bending moment and shear force. To this aim, an iterative procedure was adopted where, taking into account the gravitational loads of the deck and the piers, a funicular arch was generated starting from the shape of a parabolic arch [2]. This was a common practice in the past, mainly in order to facilitate the construction of the bridge; if the vault was built first and, later, the entire superstructure was built on it, it is possible to use very light scaffolding for the vault during the hardening of concrete. In fact, after this phase, the arch alone is able to bear the loads for the next phases of the construction.

The length of the bridge, at the imposts of the arch, is $L = 60$ m, while the rise is $f = 24$ m; the deck is 9 m wide (Figure 2b) and it consists of one driveway with two lanes (3.5 m each) and with two lateral sidewalks 1 m wide. The structure of the deck is made of a regular grid of main and secondary RC beams (Figure 2b) with rectangular cross sections with dimensions of $0.3\,\mathrm{m} \times 1.2\,\mathrm{m}$ and $0.3\,\mathrm{m} \times 0.6\,\mathrm{m}$, respectively, and of a slab 0.2 m thick. A total of 11 triplets of piers with a constant 0.3 m × 0.5 m cross section connects the vault to the deck (see Figure 2a).

Typical concrete and steel usually used for this type of structure in ’50s of 20^{th} century were assumed. Based on historical sources and on several consulted projects of the time, TOR 60 steel was largely used: it was characterized by an average yield

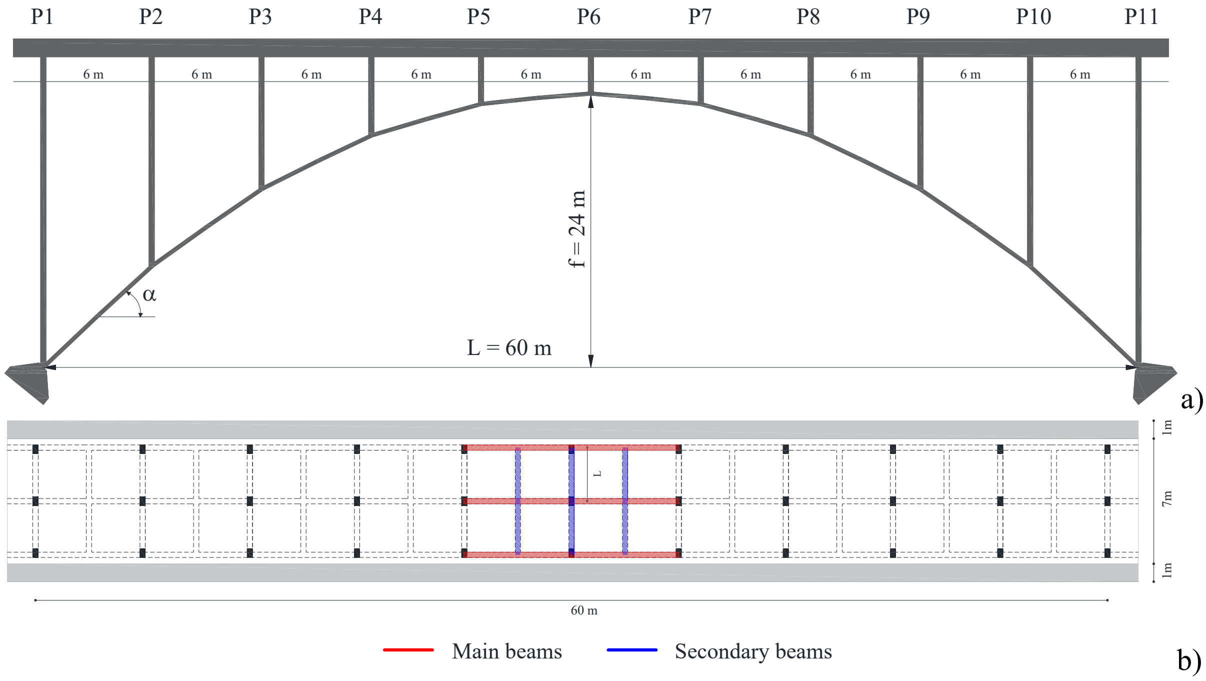


FIGURE 2. a) Longitudinal view of the deck arch bridge assumed as case study; b) Plan view of the deck.

point of about 600 MPa and was classified as semi-hard or hard steel. A characteristic cylindrical compressive strength $f_{ck} = 30$ MPa was adopted (i.e. about an average cubic strength $R_{cm} = 45.8$ MPa), since semi-hard steel was usually adopted with more performant concretes. The structural elements of the bridge were designed replicating the common practice of the past. At that time, very simple structural schemes were adopted in comparison with current design approaches. For the purposes of this work, a simulated design was performed only for the beams, the piers and the arch. Based on the design rules of that time and after the examination of old projects of RC deck arch bridges too [3], the typical design procedure was replicated in order to define the dimensions of elements and amount of steel reinforcement.

After the category of the bridge is fixed depending on the road typology, the expected traffic load was defined. Thus, the analysis of load combinations was carried out with reference to the mandatory codes of the time, in particular the Royal Decree of 1939 [4] and the Circular n. 6018 on the loads on bridge structures of 1945 [5]. In particular, assuming a bridge of first category (high traffic roads), two different types of loading schemes were adopted:

1. Two or more undefined trains of trucks weighing 12 tons (about 120 kN) side by side;
2. Two compressor rollers weighing 18 tons each (about 180 kN) side by side.

In addition, a compact crowd was expected on the sidewalks next to the driveway.

The second load scheme was typically considered for local verifications, since it locally provides higher stresses (a concentrated force of 6 tons per wheel was, indeed, assumed). Conversely, the design of the main structural elements - i.e. piers, arch, and beams - was carried out with reference to the first load scheme, which globally engages the entire structure. It is worth to note that in the following, the simulated design will be carried out according to a simplified model and will be, thus, focussed only on the beams, piers and arch as load carrying elements, while the slab of the deck will be considered only with its weight.

2.2. Assessment of loading schemes for the main beams

There are many simple structural schemes that can be used for a preliminary design. The approach adopted in this paper considers an equivalent two-dimensional structure, characterized by a continuous deck beam supported by the piers, a fully fixed arch at its imposts and axially rigid piers connected to the deck and the arch with hinges at both ends, as commonly adopted by the designers of that time. Such an assumption yields to zero shear and bending actions on the piers, while the real constraints and the detailing allows for the development of some shear and bending actions; however usually this mismatch has negligible impact on the structural behaviour. Moreover, in this phase, the main beams are considered rectangular instead that T shaped, hence neglecting the structural contribution of the slab, but with the height equal to the whole deck thickness.

For the main beams, the assessment of maximum stress was carried out by placing the moving loads, given by the loading condition 1 [4, 5], in the most

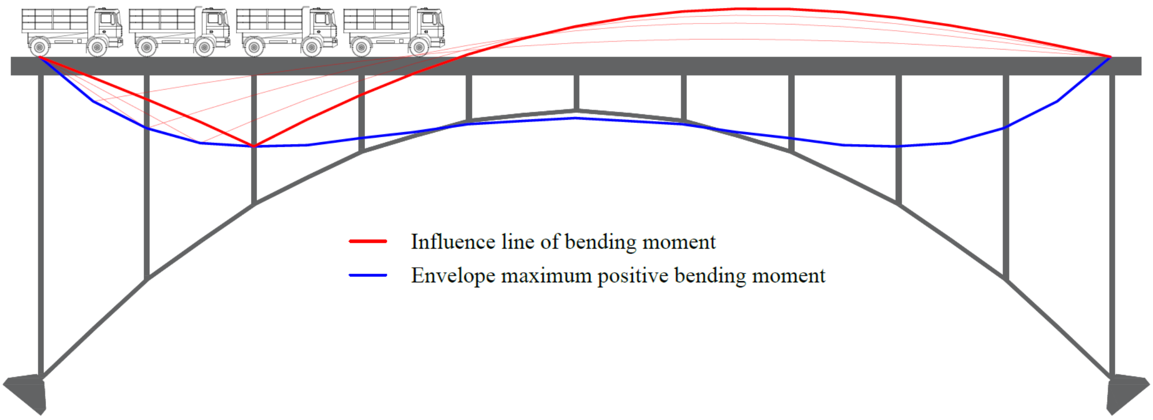


Figure 3. Influence line of bending moment and position of the moving loads able to maximize the positive bending moment.

unfavourable position on the deck, which was found by means of the Influence Lines. Figure 3 shows the influence lines of the bending moment for moving vertical loads in some cross sections of the deck (red lines). The envelope of the peaks (blue line in Figure 3) provides the diagram of the maximum and minimum moments from which it is possible to identify the cross section and, therefore, the position of the moving loads able to maximize the bending moment. It was found that the most stressed section is at about 1/4 of the bridge span and the loads associated to its development should be applied on about one third of the bridge span.

Given the maximum moment (positive and negative), it has to be divided among the individual main beams of the deck. The division can follow different approaches: for the examined case, it was assumed that the load solidly affects all the elements of the deck so that each beam is burdened by the same percentage of load. It is worth to note that the maximum bending moment induced by the moving loads must be combined with the actions produced by the gravitational loads; to take into account the dynamic effects, the moving loads have been amplified by 35%, which is obtained by the following formulation:

$$\varphi = 1 + \frac{16}{L + 40} \tag{1}$$

where L is the span of the main beams to be designed.

With regard to the Royal Decree of 1939 [4], the design of steel reinforcement was made according to the "Allowable Stress Method (ASM)", also known in literature as "Working Stress Method (WSM)" [1]. For the main beams, the steel bars have been designed with the simplified formula for RC cross section with single bars layer:

$$A_s = \frac{M}{0.9\, d\, \overline{\sigma_{y,\,k}}} \tag{2}$$

where d is the effective height and $\overline{\sigma_{y,\,k}}$ is the steel working stress, which is established by means of a safety factor 2 with respect to the characteristic yielding stress and, thus, is 300 MPa, being $\overline{\sigma_{y,\,k}} = 600$ MPa.

Moreover, for the main beams, a double steel bars layer has been placed, a common solution found in several old projects [3]. Details of steel reinforcement for the main beams are reported in Figure 5.

2.3. Assessment of loading schemes for the arch, the piers and secondary beams

The position of the moving loads giving the maximum bending moment on the deck does not yield the maximum horizontal thrust in the arch too. In fact, in order to maximize the horizontal thrust, H, and the compression force, N, in the arch as well, the moving loads must be applied on the whole deck. Since the arch is designed to be funicular of self-weights and even under moving loads it remains funicular due to the high flexural stiffness of the deck (because bending moments in the arch remain negligible), the compression force in the arch can be calculated by knowing the value of the horizontal thrust H and inclination of cross section, α:

$$N = \frac{H}{\cos\alpha} \tag{3}$$

The piers were studied under the same load condition considered for the arch. It is worth to note that both the piers and the arch, under the simplified design assumption, were designed to carry compression force only. Under such an assumption, the area of the steel bars was designed as 0.8% of the ideal concrete

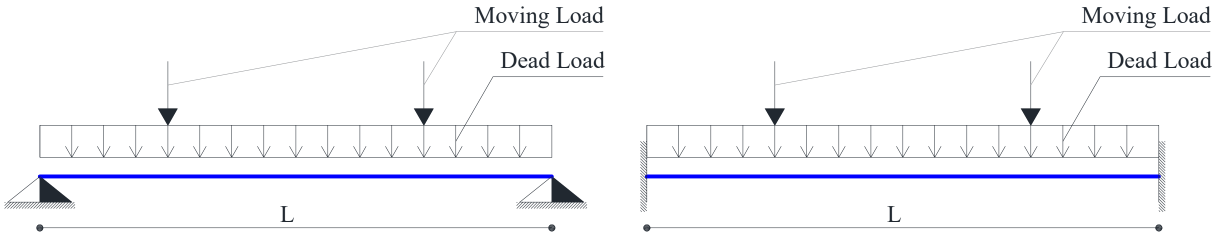


Figure 4. Different constraint conditions for the secondary beams.

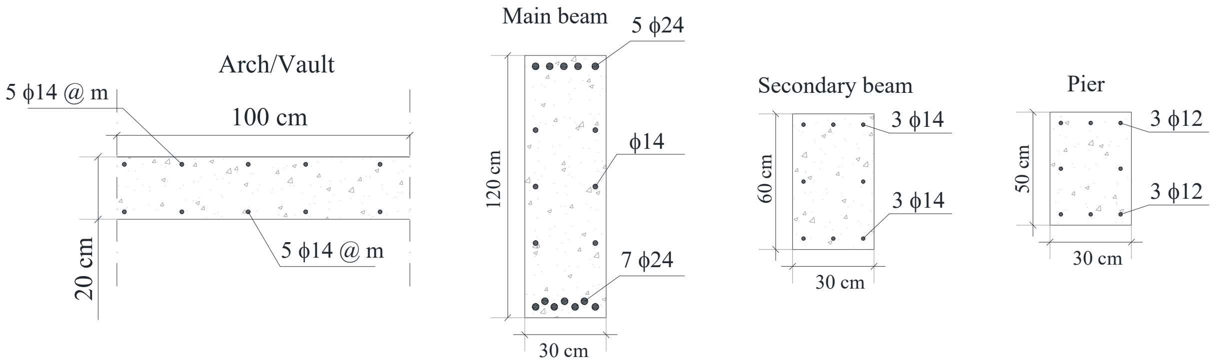


Figure 5. Details of cross section and steel reinforcement of different structural elements.

area, i.e. the minimum area of concrete related to the maximum compressive force, N [4]:

$$A_{ci} = \frac{N}{\overline{\sigma_{c,\,c}}} \tag{4}$$

where $\overline{\sigma_{c,\,c}}$ is the working stress for the concrete under compression which was assumed equal to 70% of $\overline{\sigma_{c,\,f}}$.

Since the compressive cylinder strength of concrete was assumed as $f_{ck} = 30$ MPa (i.e. characteristic cubic compressive strength $R_{ck} = 36$ MPa), the working stresses under flexural action, $\overline{\sigma_{c,\,f}}$, is:

$$\overline{\sigma_{c,\,f}} \;=\; 7.5 + \frac{R_{ck} - 22.5}{9} \;=\; 9\,\text{MPa} \tag{5}$$

and the maximum working stress in the concrete under compression forces is, thus, $\overline{\sigma_{c,\,c}} = 6.3$ MPa.

The secondary beams were considered appropriately fixed to the main beam. In particular, the constraint conditions at the ends have been assumed as ranging between a simple supported beam and a fully fixed beam, in order to maximize the positive and negative bending moments along the element (Figure 4a).

The secondary beams have been designed with reference to their tributary area to define the entity of the gravitational loads (uniformly distributed loads) and with reference to the worst condition for the moving loads, i.e. when the axis of the vehicle is just on the secondary beam (loading condition 2 according to [4, 5]). The steel reinforcement was designed according to Eq. 2 too.

Figure 5 reports the details of cross sections and steel reinforcement for the arch, the piers and the beams.

3. Analyses and structural verifications

Based on the bridge geometry discussed in the previous section, a three-dimensional model has been implemented in the SAP2000 software [6]; two-node frame elements are used to model the beams of the deck and the piers of the bridge. The eccentricity of the piers with respect to the axis of the arch is taken into account with rigid offset and massless links. Differently from the simplified schemes adopted in the simulated design, the piers are fixed at the ends and are not connected, thus, by hinges.

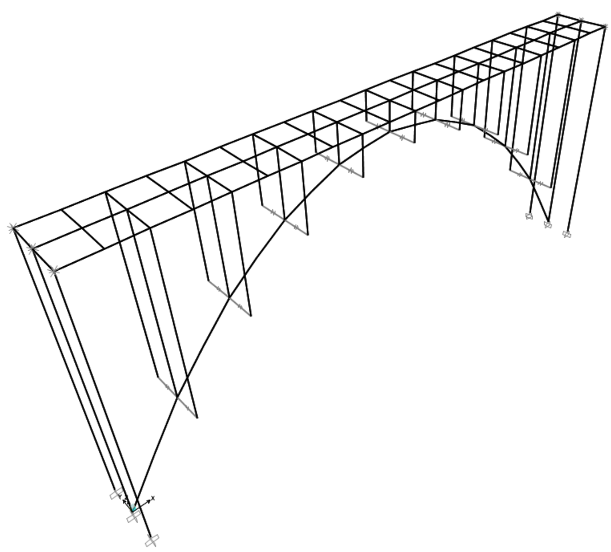


Figure 6. 3D FE model of the RC deck arch bridge assumed as case study.

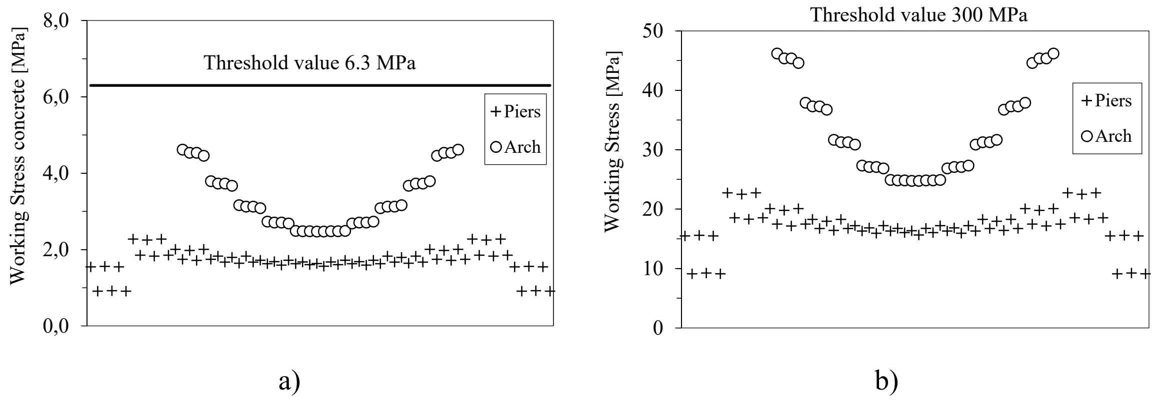


Figure 7. Structural verifications of compression stresses in the arch and the piers: a) compressed concrete; b)ăcompressed steel reinforcement.

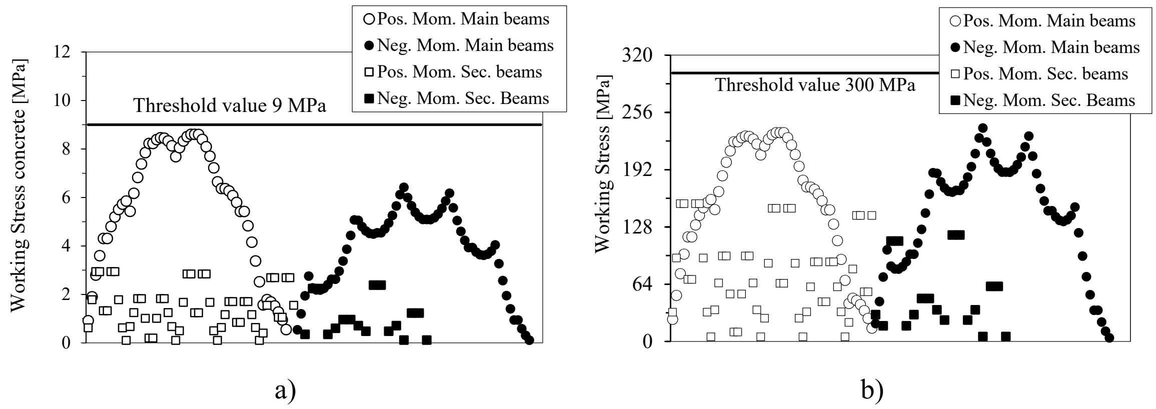


Figure 8. Structural verifications for bending moment in the beams: a) compressed concrete, b) steel reinforcement in tension.

Based on the FE model shown in Figure 6, the simple RC deck arch bridge assumed as case study was analysed under the loading conditions prescribed by Circular n. 6018 of 1945 [5]. In order to carry out a global analysis of the structure, the undefined trains of trucks weighing 12 tons were considered, i.e. the loading condition 1 provided by [5]. Such a loading scheme was converted into a uniform distributed load by means of appropriate tables [5]. Each main beam, in addition to its own weight and the loads relative to the pavement, was loaded by the same amount of moving loads, according to a common assumption particularly widespread in ’40s and ’50s years. The compact crowd, on the other hand, was considered only for the two main edge beams. The structural verifications, as done for the simulated design, have been performed according to the WSM [1]. In addition to the allowable normal stresses under bending actions for the concrete given by Eq. 5, the allowable shear stresses, for a concrete with a good performance, were equal to:

$$\tau_{c,0} = 0.6\,\text{MPa},\ \ \tau_{c,1} = 1.6\,\text{MPa} \qquad (6)$$

Note that if $\tau < \tau_{c,0}$, it is not necessary to design an adequate shear reinforcement and usually 3 or 4 stirrups per meter were adopted. If $\tau_{c,0} < \tau < \tau_{c,1}$ an adequate shear reinforcement is required; otherwise, it is necessary to change the RC cross section.

For the steel, the same allowable stress of 300 MPa has been considered.

According to the WSM, the structural verifications are satisfied when the stresses in all the cross sections of the element are lower than the allowable thresholds, assuming a linear elastic behaviour for materials and a homogenization coefficient $n = 10$ [4]. Figure 7 summarizes the working stresses in compression in the concrete and in the steel reinforcement distributed along the horizontal axis for the cross sections of the arch and the piers under compression forces. The working stresses are significantly lower than the threshold values (6.3 MPa for concrete and 300 MPa for steel).

Moreover, the working stresses in the beams, both main and secondary ones freely distributed along the horizontal axis, under bending moment (positive and negative) and shear forces are shown in Figure 8 and 9, respectively. Figure 8 highlights that the stresses in every cross section due to bending moments are always lower than the allowable ones (9 MPa for concrete and 300 MPa for steel). Moreover, Figure 9 shows that in every cross section the shear stresses are always lower than $\tau_{c,0}$, meaning that it was not

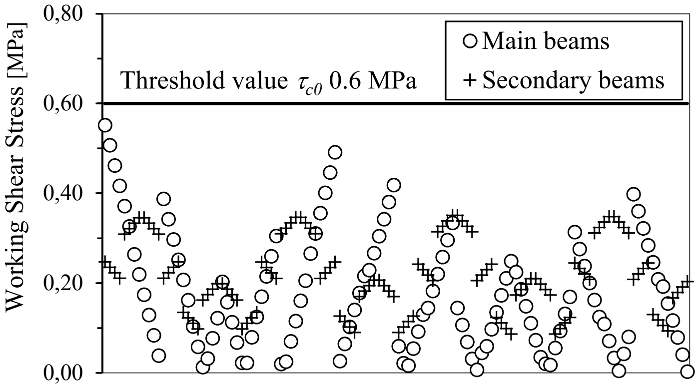


FIGURE 9. Structural verifications for shear in the beams..

necessary [4] to design a specific shear reinforcement, but it was sufficient to consider a minimum amount, i.e. 3 or 4 stirrups per meter.

It is worth noting that the shear verification was carried out only for the beams since the arch, designed as funicular of the self-weight of the whole structure, and the piers, designed as pendulums, are expected to work mainly in compression. The FE model in SAP2000 has, indeed, highlighted the presence of bending moments and shear actions on the arch as well, although of very small entity if compared to the dominant axial compression load. Although starting from a very simple structural scheme, it seems that the actions on the structural elements calculated with a more refined FE model are consistent with the simplified approaches used in the past. It is worth to remember that the WSM entails the designer to check that the maximum working stresses in the structural elements under the service loads do not exceed the allowable stress. Therefore, although some values are close to the reference thresholds, it can be reasonably supposed that they are however far from the effective failure values. Future analyses will be, indeed, focussed on the verification of the same case study under the actions provided by current Italian building code and adopting the current methodology, i.e. the Limit States approach.

4. Conclusions

Several RC deck arch bridges were built around the 1950s in Italy. The present work analyses the simulated design of a RC "Maillart" arch bridge, in particular, replicating a typical approach adopted in the past. Starting from simple design schemes, a simulated design was performed accounting for the load conditions required by the past codes. The response of the case study was, then, examined by a 3D finite element model under the same load conditions adopted in the simulated design phase, where real shape and constraints have been simulated. The structure was verified according to the Working Stress Method (WSM), typical of the 1950s. All the verifications resulted adequate.

Further studies will investigate several modelling strategies and will concern the analysis of the case study under the actions provided by new Italian building code and performing the verifications according to the current verification methodology, i.e. the Limit States approach.

References

[1] W. Lin, T. Yoda. Bridge Engineering Classifications Design Loading and Analysis Methods, Butterworth Heinemann, 2017. :https://doi.org/10.1016/B978-0-12-804432-2.00001-3.

[2] V. Franciosi. Il Ponte ad Arco (Liguori Editore), 1994.

[3] W. D. Ponti. Italiani in Cemento Armato. Santarella and Miozzi (Book Review). *Town Planning Review* **15**(3), 1933. https://doi.org/10.3828/tpr.15.3.hv41614521888141.

[4] Regio Decreto 16/11/1939 n. 2229 Norme per la esecuzione delle opere in conglomerato cementizio semplice ed armato (in italian), 1939. https://www.edilportale.com/normativa/regio-decreto/1939/2229/norme-per-la-esecuzione-delle-opere-in-conglomerato-cementizio-semplice-od-armato_15060.html.

[5] Ministero dei lavori pubblici: Direzione generale dei servizi tecnici. Circolare 9/06/1945 no. 6018 Carichi da considerare nel calcolo di ponti per strade ordinarie (Ministero dei lavori pubblici - Direzione generale dei servizi tecnici) (in italian), 1945.

[6] Computer & Structures Inc. (CSI) SAP2000 Integrated Software for Structural Analysis and Design Computers and Structures Inc., Berkeley, California, 2013. https://www.scirp.org/(S(351jmbntvnsjt1aadkposzje))/reference/ReferencesPapers.aspx?ReferenceID=961255.