Structural Analysis of Historical Constructions – Jerzy Jasieńko (ed)
© 2012 DWE, Wrocław, Poland, ISSN 0860-2395, ISBN 978-83-7125-216-7

AN OVERVIEW ON ITALIAN RAILWAY MASONRY BRIDGES WITH LOAD-CARRYING CAPABILITY ESTIMATE

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ABSTRACT

Italian rail network includes a wide number of masonry arch bridges. Most of them are currently in service but a deep awareness of their actual safety level is still lacking. The load carrying capability of a representative sample of Italian large-span railway masonry bridges is assessed in the present paper. The bridges date back to XIX and XX Centuries and vary in terms of geographical position and geometric properties. Nevertheless, the main design and building characteristics recur all over the national territory and comply with the criteria suggested in historical treatises from Curioni, Perronet, Séjourné, Gautier and other authors. On the whole, 34 bridges are analysed under different constitutive assumptions and the reliability of yield-design based approaches is discussed. The ultimate loads are related to the main geometric characteristics, such as the shape of the arch, the slenderness of the piers and the number of spans, to identify the most vulnerable bridge types. The effect of the variation of mechanical and geometric parameters that are generally difficult to be precisely determined, is also studied.

Keywords: *Masonry arch bridges, Load carrying capability, Railway bridges, Constitutive assumptions, Historic design criteria, Geometric issues*

1. INTRODUCTION

Italian railway network comprises a wide heritage of masonry arch bridges, ranging from small single-span overpasses to large multi-span viaducts. Most of the historical bridges are almost coeval, being built in the second half of XIX Century and in the first decades of XX Century, and, even if geographical position, materials and dimensions may change, similar geometries and building techniques can be widely recognized. The recurring analogies can be traced back to the design criteria suggested by historical treatises and manuals [1-18], which were based on graphical procedures, such as the Mery's method [12], or on empirical methods and have been diffused all over the national territory after the unification of Italy (1860s). Some of the most representative examples are reported in Table 1 and Table 2 for the arch thickness and the pier top width, respectively.
Concerning this, it is worth pointing out that the study of construction technologies and design methods is widely acknowledged not only as a precious contribution from a cultural perspective, but also as a fundamental starting point in the assessment of an existing construction [19-22], since it provides useful information on its internal structure (including foundations and structural details), building processes (which may in turn influence the internal stress distribution), adopted materials.
Despite traffic loads have significantly increased and material deterioration processes, foundation settlements, damage development, transformations or partial demolitions may have occurred with the passing of time, a deep investigation on the actual safety level of existing masonry bridges under the exercise conditions provided by current design codes is strongly needed.

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Table 1 Historical design rules for arch thickness
s: thickness of the arch crown; S: span; R: radius; a: Skewback angle

Year	Author	Deep arch	Shallow arch
1716	Gautier	$s = 0.32 + S/15$	–
1788	Perronet	$s = 0.325 + 0.035S$	$s = 0.325 + 0.0694R$
1809	Gauthey	$s = 0.33 + S/48$ $(S < 16m)$	–
		$s = S/24$ $(16m \leq S < 32m)$	–
		$s = 0.67 + S/48$ $(S > 32m)$	–
1809	Sganzin	$s = 0.325 + 0.03472S$	–
1845	Déjardin	$s = 0.30 + 0.045S$	$s = 0.30 + 0.025S$
1854	L'Éveillé	$s = 0.333 + 0.033S$	$s = 0.33 + 0.033\sqrt{S}$
1855	Lesguillier	$s = 0.10 + 0.20\sqrt{S}$	$s = 0.10 + 0.20\sqrt{S}$
1862	Rankine	$s = 0.19\sqrt{R}$	–
1865	Curioni	$s = 0.24 + 0.05S$	$s = 0.24 + 0.07R$ $(\alpha < 45°)$
			$s = 0.24 + 0.05R$ $(\alpha < 60°)$
1870	Dupuit	$s = 0.20\sqrt{S}$	$s = 0.15\sqrt{S}$
1885	Croizette-Desnoyers	$s = 0.15 + 0.20\sqrt{R}$	–
XIX Cent.	Udine-Pontebba railway	$s = (1 + 0.10S)/3$	$(1 + 0.20R)/3$
1914	Séjourné	$s = 0.15 + 0.15\sqrt{S}$	
1926	Breymann	2 brick heads (24 cm) $(S < 1.75$ m$)$	1 brick head more than for deep arches
		3 brick heads (36 cm) $(2$ m $< S < 3$ m$)$	
		4 brick heads (48 cm) $(3.5$ m $< S < 5.75$ m$)$	
		5 brick heads (60 cm) $(6$ m $< S < 8.5$m$)$	
		$s = S/15 \div S/12$ $(S > 8.5$ m$)$	

Table 2 Historical empirical rules for the thickness of the pier top from different authors
s: thickness of the arch crown; S: span; P: thickness of the pier top section

Year	Author	Deep arch	Shallow arch
1684	Blondel	$P = S/4$	$S/4 \leq P \leq S/3$
1716	Gautier	$P = S/5$	
1788	Perronet	$P = 2.25$ s	
XIX Cent.	German engineers	$P = 0.292 + 2$ s	
1881	Rofflaen	$P = 2.5$ s $(10$ m $> S)$	
		$P = 3.5$ s $(10$ m $\leq S)$	
1914	Séjourné	$S/10 \leq P \leq S/8$	

In the current paper, the load carrying capability of a representative sample of 34 Italian railway masonry arch bridges is assessed. Different bridge types are included in the sample such as deep and shallow arches, thin and thick vaults, slender and squat piers, multi-span and single-span bridges. The results are related to the main geometric issues to identify their influence on the structural safety. The effect of the constitutive assumptions on the overall strength is also studied and the reliability of yield-design based approaches is discussed. Finally, the sensitivity to the variation of some parameters that are generally difficult to be precisely determined, such as the crushing strength and the ductility of masonry, the arch thickness and the backfill height, is investigated.

2. BRIDGE SAMPLE AND MODELLING APPROACH

The railway masonry arch bridges studied in the current work are represented on the Italian territory in Fig. 1 in which their names and construction years are also collected. Drawing on [22], the arch thickness at the crown and the pier width are plotted in Fig. 2 superposed to the historical empirical rules showing that nearly all the bridges comply with the classical design criteria. The vault thickness design was related to the span, using different criteria for deep and shallow arches (Fig. 2a and 2b). Based on the considered sample, the pier section was dimensioned starting from the span (Fig. 2c), rather than from the vault thickness (Fig. 2d).

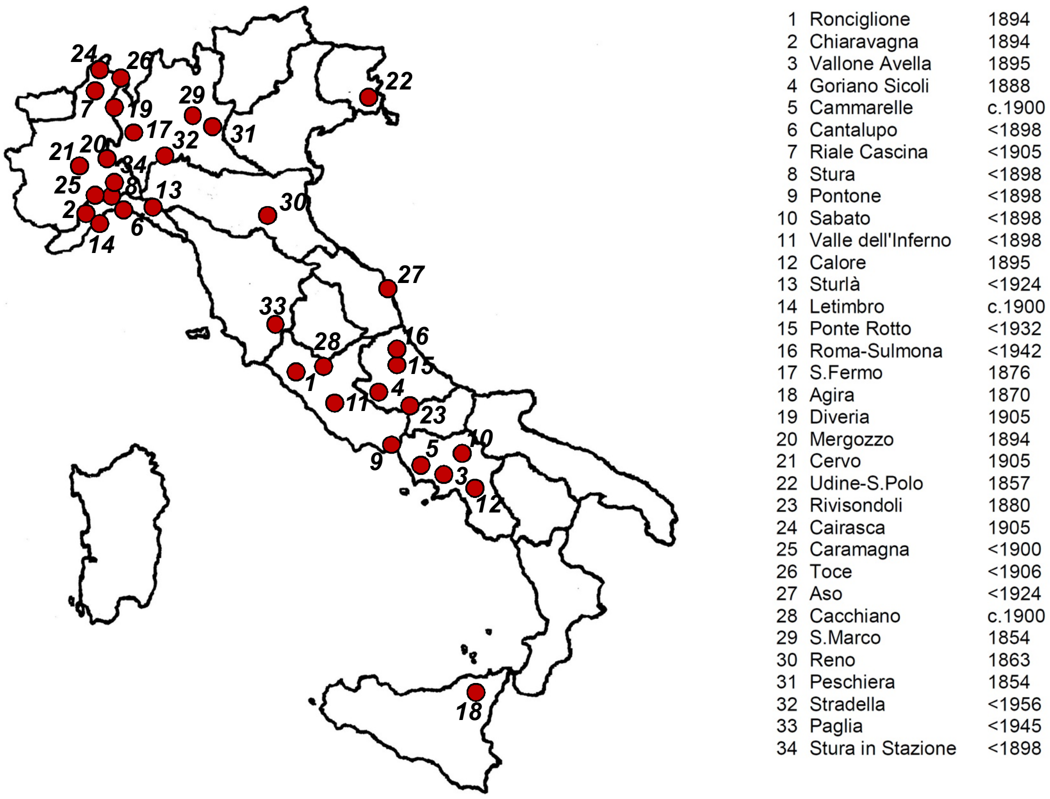

Fig. 1 Sample of bridges on the Italian territory

Starting from experimental results showing that plane sections remain plane after deformation [23], masonry elements under compression and bending can be represented through 1D models. Based on this approach, in the current work bridge arches and piers are represented as segmental beams, made out of a series of flexibility based beam elements whose cross section is discretised into fibres [24, 25]. The constitutive characterization is made by assigning to the fibres uniaxial stress-strain relations. The abutments and the backfill are represented by non-linear fibre truss elements, resisting only in compression. As for the fill soil, its self weight is accounted for, together with the load spreading effect from the track level to the arch extrados and, to this purpose, a diffusion angle of 40° + 40° is assumed.

The method allows the material properties to be described in detail. Accurate analyses under any loading regime can be performed, ensuring at the same time low computational costs thanks to the intrinsic simplicity of a frame element and to the discretisation of the cross section into fibres. On the other hand, some simplifying assumptions are made. The effect of spandrels and fill soil in terms of stiffness and strength is neglected, which may lead to an underestimate of the actual load carrying capability. Also, the abutments are considered as perfectly fixed, which may instead yield to an overestimate of the effective strength.

The travelling loads are applied within an incremental procedure and the maximum value is evaluated for each position they assume along the bridge deck. The load carrying capability curve is hence built and the safety factor (SF) is defined as the ratio between the maximum load resultant and the exercise load provided by the standard code [26] for rail bridges. LM71load model is adopted, consisting of four point forces of 250 kN each. The design exercise load value is therefore 1000 kN per line running on the bridge. Aiming at being on the safe side, the distributed load is not included in the load pattern, since it constrains the upwards deflection of the arches close to the loaded span and prevents the development of the collapse mechanism in a multi-span arch bridge [25], which in turn leads to a higher load carrying capability.

3. EFFECT OF CONSTITUTIVE ASSUMPTIONS

The load carrying capability of the 34 arch bridges included in the sample is assessed assuming different constitutive relations representing the local behaviour of masonry. Three uniaxial material laws are used such as elastic no-tensile (ENT), having elastic indefinite response in compression, elasto-plastic (EP), accounting for a finite compressive strength and, finally, Kent&Park (KP) [27], including both a finite strength and a limited ductility ($\eta = \varepsilon_{cu}/\varepsilon_{c0}$) (Fig. 3). While the classical mechanism method is based on Heyman's hypotheses of infinite compressive strength and no tensile resistance [28], which are represented by ENT material, yield-design based approaches (limit analysis) rely on the assumptions that masonry is elasto-plastic in compression (EP). Finally, KP relation faithfully reproduces the actual mechanical response revealed by experimental studies carried out on masonry prisms under compression and bending, including the non-linear pre-peak phase and the post-peak deterioration [23].

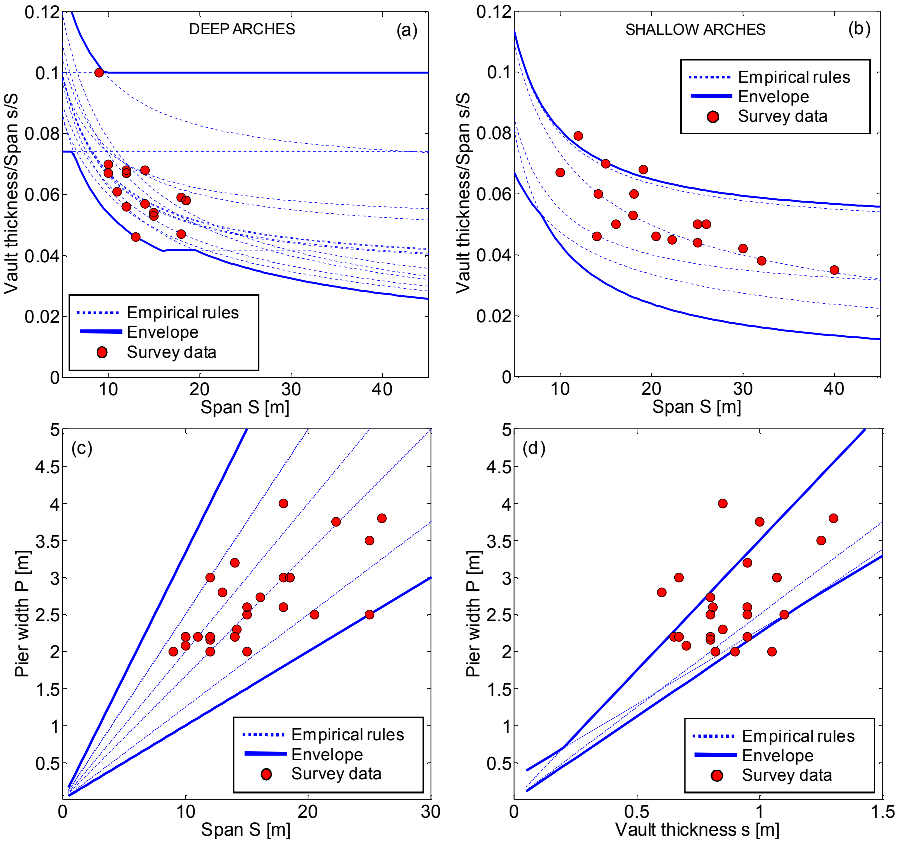

Fig. 2 Surveyed data and historical design rules: vault thickness vs. span for deep arches (a) and shallow arches (b); pier top width vs. span (c) and vault thickness (d)

The load carrying capability curves resulting from analyses under travelling loads show that a significant reduction of the overall bridge safety is found when finite crushing strength and limited ductility are accounted for. The load carrying capability curves of Cervo River Bridge (#21) and Riale Cascina Bridge (#6) are represented in Fig. 4 and 5, respectively. The former is a 5-span bridge having squat piers and shallow arches, while the latter is a 6-span bridge having slender piers and deep arches. The arch built on the highest pillars is usually the weakest one due to the relative movement of its springers, provided that this is neither the first nor the last span. Slight variations of the critical load position may however result under different material assumptions, because of the variations in the stress field induced on the structural elements by the increasing load within the incremental analysis. The crown is the most critical position for KP and EP materials, since it is related to the highest stress resultant in the arch cross section, while when the crushing failure of masonry is neglected (ENT) asymmetric load configurations become the most severe ones and the collapse mechanism is activated when the load resultant is at about 0.35 S.

As for the whole sample, analyses performed with KP material lead to safety factors between 1.3 and 13.0, indicating that the considered bridge stock is safe under the design exercise loads. Fig. 6a collects the dots from all the analyses. The effect of an accurate description of the material properties (crushing strength, post-peak behaviour) clearly results, indicating how important it may be when assessing the load carrying capability of a masonry arch bridge.

Despite a certain scatter, due to the variability of the considered sample, a general trend can be identified. The overestimate provided by the simplified constitutive assumptions is, on average, 94% and 33% for ENT and EP relations, respectively. The coefficient of determination R^2 is lower for ENT material (0.66) than for EP (0.88), indicating that a worse linear correlation is found in the former case. The higher data dispersion is likely to be related to some extremely high values of SF (more than 20) obtained in shallow and squat arch bridges when the crushing strength is assumed to be infinite. This is confirmed by the strong relation between safety factor overestimate under simplified constitutive assumptions and rise-to-span ratio (Fig. 6b) as it influences the stress level within the arch section. The other relations between safety level and geometrical issues are described in detail in the following section for EP and KP materials.

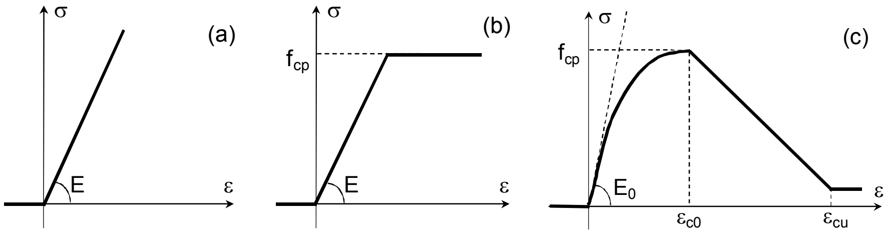

Fig. 3 Constitutive laws adopted for load carrying capability analyses: ENT (a), EP (b), KP (c)

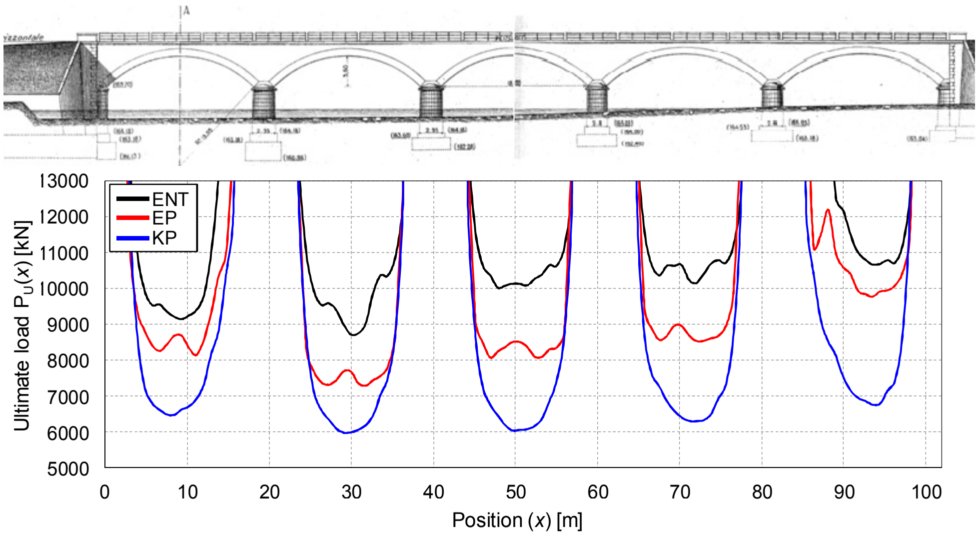

Fig. 4 Load carrying capability of Cervo River Bridge under LM71 load model for different constitutive assumptions

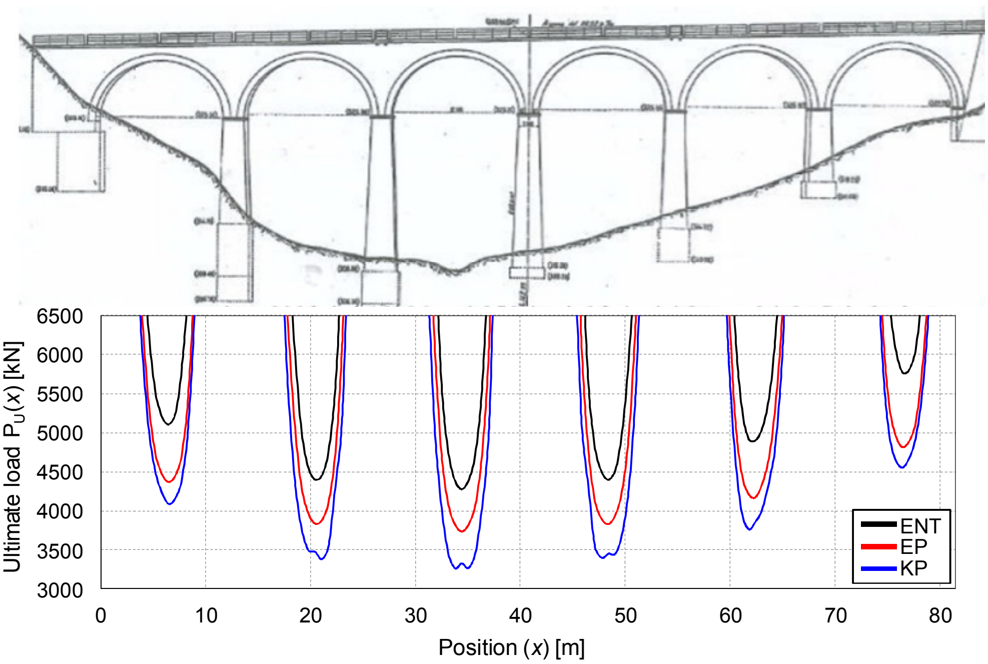

Fig. 5 Load carrying capability of Riale Cascina Bridge under LM71 load model for different constitutive assumptions

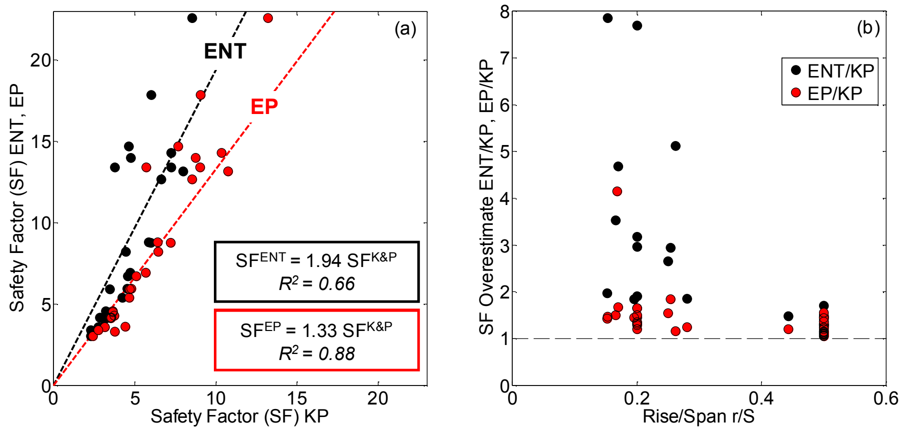

Fig. 6 Safety factor under different constitutive assumptions: ENT and EP vs. KP (a) and ENT/KP and EP/KP ratios vs. rise-to-span ratio (b)

4. EFFECT OF GEOMETRY

It is relatively difficult to relate the safety factor to the geometrical properties since numerous issues play an important role in the structural response. Some of them have however a higher influence, such as the span, the vault thickness, the rise-to-span ratio, the number of spans and the maximum pier height. Their effect on the safety level is shown in Fig. 7 for EP and KP material relations. Based on the sample considered in the current work, bridges with larger span result to be safer (Fig. 7a), even if this is probably due to the corresponding higher vault thickness (Fig. 7b). Shallow arch bridges (low r/S) are found to be safer than deep arches (Fig. 7c), while no significant correlation is found between safety and slenderness (s/S) (Fig. 7d).

Single arch bridges and bridges with low piers are safer, as it is shown by the graphs in Fig. 7e and 7f. When one or more piers are involved in the collapse mechanism, the resulting ultimate load is lower than when the only arch fails and neither interactions between arches and piers, nor between adjacent spans, occur. The safety factor globally decreases with the increase of the number of spans but no strong variation is found for more than 3 spans, since, at the most, the loaded span and the two adjacent ones are involved in the failure mechanism.

As for the pier height, the maximum value is taken as the most significant parameter, since the weakest span is generally the one built on the highest pillars. Concerning this, it should be observed that the empirical rules do not consider any interactions between arch and pillar nor between adjacent spans. The arch is dimensioned regardless of the eventual presence of piers and, let alone, their height.

5. UNCERTAIN PARAMETERS

The structural analysis of a masonry arch bridge requires the knowledge of the main geometrical properties of the structure as well as the mechanical characteristics of the material. Some of them, however, cannot be clearly deduced from a visual survey and need to be assumed on the base of the knowledge of similar structures, if original drawings and design documents are not available or deep inspections and field testing activities cannot be carried out.

An accurate analysis of the effect of the uncertainties on the load carrying capability would need a probabilistic approach in which they are treated as stochastic variables [29]. A rough estimate of the role played by uncertain parameters can be however obtained by sensitivity analyses within a reasonable range around the most probable value.

The sensitivity to material crushing strength (f_c) and ductility (η) as well as to vault thickness (s) and backfill height (H_R) are considered in this section. To this purpose, 6 bridges are taken from the sample, showing similar span, number of spans and pier height to reduce variability. Nevertheless, the slenderness (s/S) ranges between 0.49 and 0.68; three bridges are shallow arches with rise-to-span ratios ranging from 0.17 to 0.28, while the others are deep arches (r/S = 0.50), to get information about the effect of arch geometry.

The results are collected in the plots in Fig. 8, having the uncertain parameter on the x-axis and the safety factor the y-axis. The former is divided by the reference value (f_{c0}, η_0, s_0, H_{R0}), which is derived from the available documentation and is the same assumed in previous analyses. The latter is divided by SF_0, defined as the safety factor obtained when the parameter assumes its reference value. Note that shallow arch bridges are represented by blue lines, while deep arches are indicated by red lines.

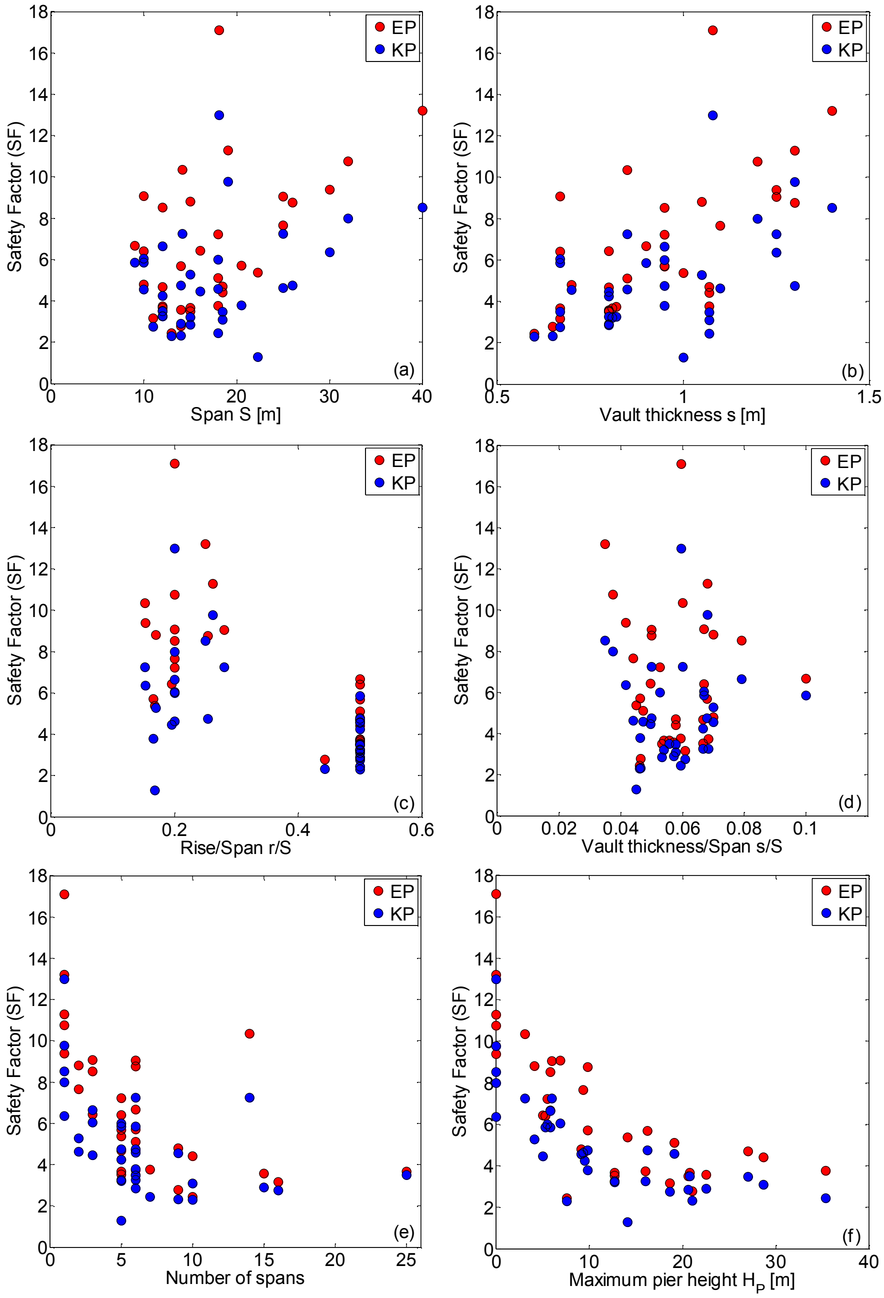

Fig. 7 Dependence of the safety factor on the geometrical properties of the bridge: span (a), vault thickness (b), rise-to-span ratio (c), thickness-to-span ratio (d), number of spans (e), maximum pier height (f)

When the crushing strength of masonry grows, the overall bridge capability grows too until it becomes so high that failure occurs due to the activation of a collapse mechanism which is independent from the material resistance (Fig. 8a). This happens for lower strength values ($f_c = 2\text{-}3f_{c0}$) for deep arches with respect to shallow arches, in which the stress level in the vault cross section is higher and the structural safety is more sensitive to the material properties. Similarly, the ductility increase is related to a higher bridge resistance until the same value of limit analysis (EP constitutive law) is reached for high ductility values ($\eta > 15\eta_0$). Shallow arches are more sensitive to the post-peak deterioration rate and the same happens also for slender arches, which is again likely to be related to the higher compressive stress state in the vault cross section (Fig. 8b).
The relations of arch thickness (Fig. 8c) and backfill height (Fig. 8d) against the bridge safety are linear and no significant variations are found between arches having different shape. It is however worth noting that for shallow arch bridges, in which the top of the backfill is relatively close to the height of the arch extrados at the crown, the safety factor grows very slowly for $H_R>H_{R0}$.

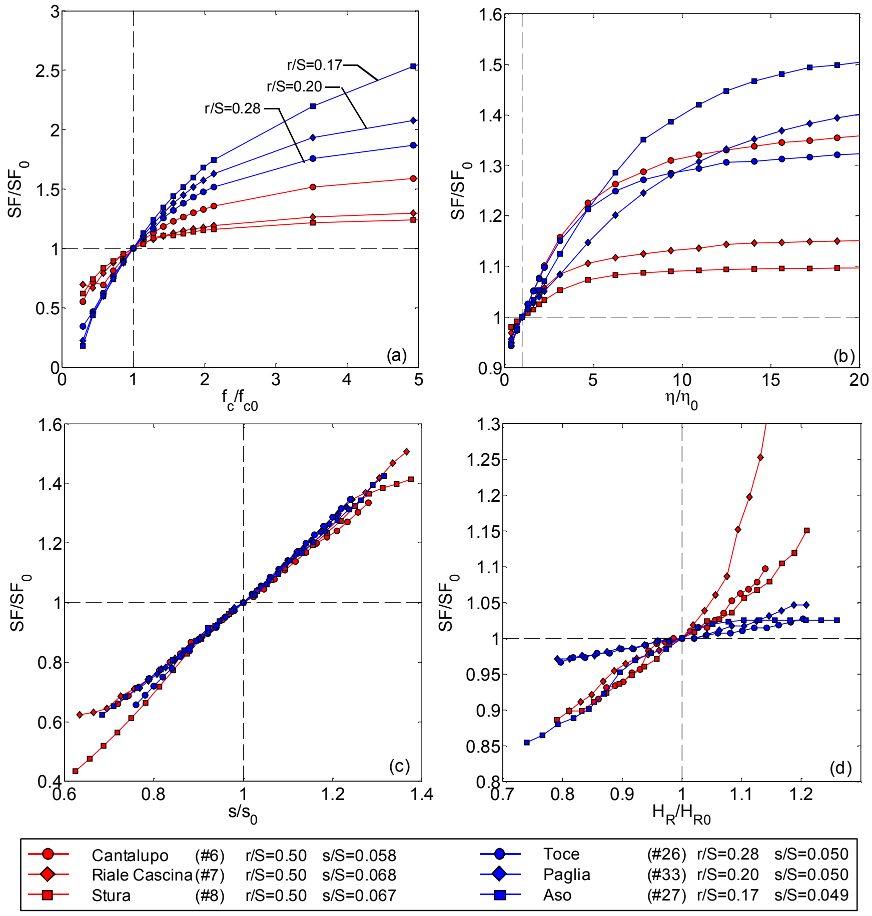

Fig. 8 Sensitivity analyses on uncertain parameters: masonry crushing strength (a) and ductility (b), vault thickness (c), backfill height (d)

CONCLUSIONS

The load carrying capability of 34 large-span Italian railway masonry arch bridges is assessed under different constitutive assumptions, revealing that an accurate description of the material properties may be of great importance. Among the considered sample, an overestimate of the safety factor in the order of 33% is found when an unlimited ductility is assumed, as in yield design methods. If the crushing strength is also neglected, as it is done in the mechanism method, such an overestimate is found to be 93%. The overestimate provided by yield design based approaches, however, does not result in an inadequate safety level of existing bridges. It may be partially balanced by the effect of fill soil and spandrels which has proved by numerous authors to play a significant role in the bridge capability (see, among others, [30]) but is neglected in the modelling approach proposed in the current work.

The safety level can be related to the main geometrical issues of the bridge and a higher load-carrying capability is found for bridges with shallow arches and thick vaults, as well as for bridges made out of a single span or having squat piers. Sensitivity analyses carried out on parameters which may be difficult to be determined with precision, such as material crushing strength and ductility, vault thickness and backfill height, show that deep and slender arches are particularly sensitive to the local characteristics of masonry due to the higher compressive strength in the vault cross section, while the backfill height plays a strong role in the response of shallow arches since it constrains the arch deflection and prevents the development of the weakest collapse mechanisms.

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