

Simplified Assessment of Railway Masonry Bridges Seismic Capacity



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SUMMARY

Masonry arch bridges represent the greatest portion of all the bridges in use in the Italian railway network. The typological characteristics of these structures were identified on a large stock of 380 bridges located in high seismicity areas, subdividing them into a set of homogeneous classes.

Subsequently a parametrical analysis was performed for each class, in order to calculate the seismic capacity of the bridges: appropriate ranges for geometrical and mechanical parameters, i.e. span length, arch thickness, span-to-rise ratio, pier height, etc. were considered, and the limit horizontal load multiplier was calculated for each relevant collapse mechanism.

Finally, envelope curves representing the seismic capacity expressed in terms of limit horizontal acceleration were derived, the input data of which are only easily detectable geometric parameters. These curves can be used for a simplified vulnerability assessment of railway masonry arch bridges, and for prioritizing further investigations and retrofit interventions.

Keywords: Masonry arch bridges, Seismic assessment, Limit state analysis

1. INTRODUCTION

According to a recent survey approximately 40% of all the railway bridges in Europe are masonry arch bridges (SB-ICA, 2007), and the majority of them are more than 100 years old (over 60%, Melbourne 2007). These percentages are also representative of the Italian stock: many of these structures, mostly built in the period 1840-1930, are located on major railway lines, in high seismicity areas.

In Italy, after the enacting of the Prime Minister's Ordinance 3274/03 and its implementing provisions (Decree of the Department of Civil Protection 21/2003) the Italian public network authorities are charged not only with the routine maintenance of the bridges, but also with the seismic assessment of the strategic structures, according to the law in force. Based on an action plan implemented by RFI (the Italian Railway Network Authority), an extensive survey, aiming at a preliminary evaluation of the seismic capacity, was carried out on a stock of about 380 masonry bridges, located in zones affected by the highest seismicity levels (Zone 1 and 2 according to the national zonation map).

In this work, on the basis of an initial statistical analysis and identification of the typological characteristics of the artefacts, masonry arch bridges were subdivided into homogeneous classes of single span and multi-span structures. Subsequently, a parametrical analysis was performed for each class, considering the influence of geometrical and mechanical parameters on the structural response.

The capacity curves for longitudinal response were obtained by means of a limit analysis approach. The results were checked by non-linear static analysis, which also was used to calculate the ground acceleration corresponding to the ultimate capacity in the transverse direction. For each class, the parametrical analysis considered appropriate ranges of relevant geometrical factors such as span length, arch thickness, span-to-rise ratio, bridge width, and pier height and section, in order to calculate the correspondent horizontal load multiplier activating the collapse mechanism. Finally, a set of parametric curves representing the ultimate capacity expressed in terms of limit ground acceleration

were derived, the input data of which are purely geometric parameters directly detectable by visual inspections.

2. STATISTICAL SURVEY OF THE EXISTING STOCK

The statistical survey regarded a stock of 380 masonry bridges belonging to the Italian railway network, and is significantly representative of the entire network stock, the structures being located in different railway lines belonging to different geographical areas (see Fig. 1).

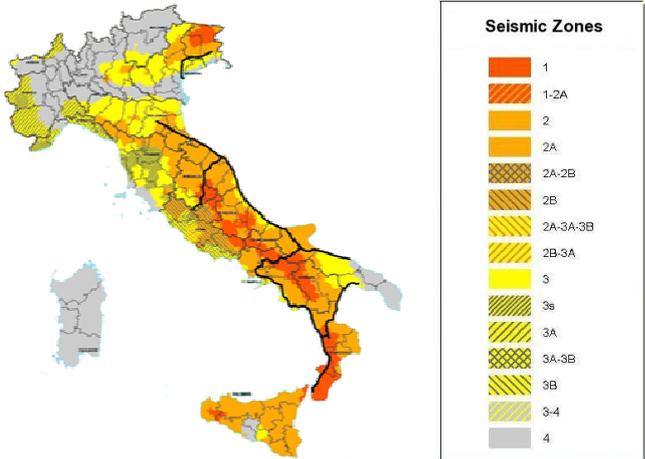
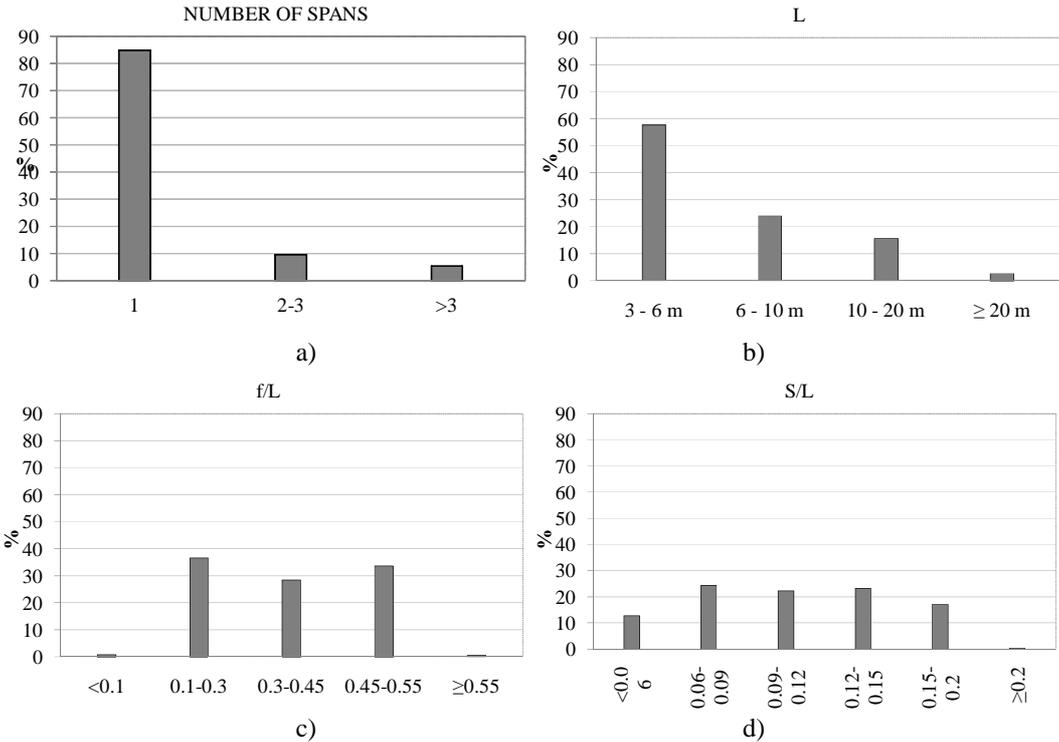


Figure 1. Plan of railway lines analyzed in the statistical survey, superimposed on the Italian seismic zonation map (seismic zonation map valid for administrative classification of the country, 2010)

As said before, these lines were chosen because they are located in Zone 1 and 2 according to the national zonation map (see Fig.1): the maximum PGA on the reference stiff soil is 0.35-0.25g for Zones 1 and 2, respectively (peak ground accelerations are computed with a 10% probability of exceedance in 50 years). Considering that masonry bridges exhibit a repetitive design, the assessable seismic risk for the masonry bridges belonging to these lines can be assumed to be the maximum expected for such structures in the entire Italian railway network.



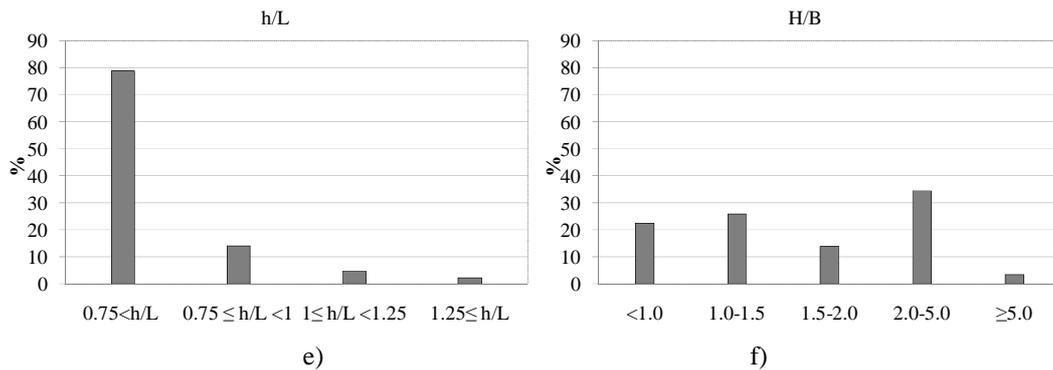


Figure 2. Statistical analysis of the railway masonry arch bridges stock. L =maximum span length, f =arch rise S =arch thickness, h =abutment height, H = pier height, B =pier longitudinal width.

As can be seen from Fig.2, most bridges are single span (85%), with short span or medium span length; a great number of arches are semicircular (35%) or depressed with high span to rise ratio ($0.3 < f/L < 0.45$), while much flattened depressed arches ($f/L < 0.15$) are generally used for medium or long spans (see also Table 3.1). The thickness of the arch varies almost linearly as a function of span length (see also Fig.4) for a certain value of span to rise ratio, their relative value (s/L) being generally higher (0.075-0.15) for short to medium span.

Among the single-span bridges, the arches with high abutments ($h/L > 0.75$) represent a not negligible portion (21%), particularly within the range of short spans ($L < 6m$). They have to be distinguished because the longitudinal collapse mechanism of the arch is different, and tend to involve the abutments too. As regards bridges with multiple spans, a parameter that greatly influences their seismic behavior is the slenderness of piers: from the statistics it can be seen that bridges with slender piers are more or less in the same percentage (48%) as bridges with squat piers.

In most cases of the analyzed stock, a single railway line or 2 lines are hosted in the bridge platform: for a single-track railway bridge the width (P) is generally 4.80-6.50m, while for a double-track bridge the width is at least 8.0m or more. The limit value $P < 8.0m$ is used in the following classification to identify narrow bridges (single-track railway bridges).

It must be specified that the data reported in Fig.2 do not include bridges with span length $L < 3m$.

3. CLASSIFICATION OF THE EXISTING STOCK

A feature of the Italian railway system is that it was largely built over a century, and that the same typologies were generally applied to all the bridge structures: this led, in masonry bridges, to the use of recurrent dimensions in structural elements and, excluding single cases due to the particular local topography, to a repetitive design, with geometrical properties varying within specific ranges, according to the Railway Manual of Practice in use.

For this reason, it is particularly suitable for a vulnerability study on a large scale, to subdivide the entire stock into homogeneous classes: in this work the classification is based on the typological features of the bridges, the geometrical ranges of the main structural elements, and the structural parameters (i.e. slenderness of piers, span to rise arch ratio, height of abutments, etc.) that mainly affect the global response, determining the type of collapse mechanism under seismic action.

As regards material properties, it was verified that that generally masonry in railway bridges is of good quality, both for the properties of the materials and for the type of the blocks laying, made with joints of adequate thickness and well organized. For this reason the range of variability of mechanical properties is quite narrow (see Tab. 3.1), and also stone masonry characteristics are not greatly different from brick masonry, the variability of the stiffness (in terms of E modulus) having an influence on the elastic phase, but not affecting significantly the ultimate behavior. In fact the tensional level in masonry arches is not so significant for the safety assessment and the resulting deformation is not relevant; conversely the ultimate resistance of arches under horizontal forces is strongly affected by geometrical parameters, the mechanism activation being characterized by the formation of three rigid voussoirs and four hinges, and the position of the hinges being directly related to the geometrical features.



Figure 3. Typical examples of masonry arch bridges, identified according to the proposed classification (Table 3.1): a) Single span with squat abutment, class 1.1 "SSsa_1.1", b) single span with high abutment "SSha_1.2", c) 2-3 spans with squat piers "2Ssq_2.1", d) 2-3 spans with slender piers "2Ssl_2.2", e) multi-span with squat piers "MSsq_3.1", f) multi-span with slender piers "MSsl_3.2"

The classification adopted relies only on geometrical factors, easily detectable by visual inspections and geometrical survey. In particular the following parameters were considered: number of spans, span length (L), arch rise (f), arch thickness (S), slenderness of piers (H/B), height of abutments (h), width of the bridge (P).

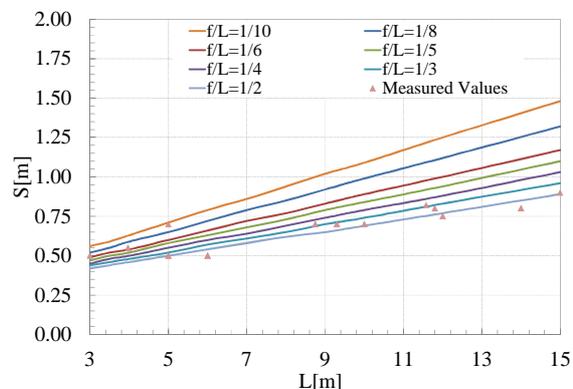


Figure 4. Values of thickness at the arch crown as a function of span length obtained from the Railway Manual of Practice, compared to the same values directly measured on a restricted set of bridges in the stock.

The ranges adopted for the parameters were defined considering typical dimensions obtained from the Railway Manual of Practice for different span length and shape of arches (semicircular arches, depressed arches with various span to rise ratios), and were compared to the ranges derived from the statistical survey, and to those obtained from a direct inspection on a restricted set of bridges (see Fig.3). The bridge classification adopted in this study is reported in Table 3.1.

Table 3.1. Hierarchical classification adopted: the macro-classes are related to the number of spans (single, 2-3 spans and multiple spans), the classes to pier or abutment characteristics, the subclasses refer to the arch parameters

MACRO-CLASS	CLASS	SUBCLASS			
		Arch Parameters			
		$L[m]$	f/L	S/L	
1) SINGLE SPAN BRIDGES (SS)	1.1) $h < 0.75L$ "Squat Abutment (sa)"	L < 6	< 0.3	[0.075-0.1[[0.1-0.15]	
		L[6-10[[0.3-0.4[> 0.4		
		L[10-20[< 0.3 [0.3-0.4[> 0.4	[0.05-0.1]	
		L > 20	[0.3-0.4[> 0.4	[0.05-0.1]	
	1.2) $h \geq 0.75L$ "High Abutment (ha)"	L < 6	< 0.3	[0.075-0.1[[0.1-0.15]	
		L[6-10[[0.3-0.4[> 0.4		
2) 2-3 SPAN BRIDGES (2S)	2.1), 3.1) $H/B < 1.5$ "Squat Piers (sp)"	P ≤ 8 m "Single Track"	L < 6	< 0.3	[0.075-0.1[
			L[6-10[[0.3-0.4[> 0.4	[0.1-0.15]
3) MULTISPAN BRIDGES (MS)	2.2) 3.2) $H/B > 1.5$ "Slender Piers (sl)"	P > 8 m "Double or MultipleTrack"	L[10-20[< 0.3 [0.3-0.4[> 0.4	[0.05-0.1]
			L > 20	< 0.3 [0.3-0.4[[0.05-0.1]

4. STRUCTURAL CAPACITY TO HORIZONTAL LOADS: KINEMATIC METHOD AND F.E. ANALYSIS

The kinematic method, based on the particularization of limit design to masonry structures proposed by Heyman (1982), is a suitable procedure for the seismic assessment of masonry arches (Clemente et al., 2010). In this study the kinematic method (KM) was used extensively in the parametric analysis to determine the acceleration limit value, which activates the collapse mechanism of the arch bridge under incremental horizontal loads. The procedure consists in an iterative approach based on the application of the principle of virtual work, the bridge being discretized into rigid voussoirs whose centroid virtual displacement is determined. The filling material is exclusively taken into account as applied mass. The assumed hypotheses are:

- absence of sliding between voussoirs (Heyman, 1982);
- infinite compressive strength of masonry (Heyman, 1982);
- large displacements (for the evaluation of the ultimate displacement at collapse with non linear kinematic analysis).

Infinitely rigid arch abutments are assumed for single span and multi-span bridges with squat abutments, where the collapse mechanism is located in the arch with the formation of 4 plastic hinges (see Figure 5). A substantially different kinematic mechanism is activated in the case of slender piers and high abutments that participate in the global mechanism (da Porto et al, 2007).

The evaluation of the arch structural capacity is articulated into as follows:

- determination of the collapse mechanism (Heyman, 1982; Clemente, 1998);

- application of the principle of virtual work to the original arch configuration, to calculate the ground acceleration that activates the collapse mechanism.

In the case of arches supported by infinitely rigid abutments and in absence of fill, Clemente (1998) provides parametric curves for that can be used to find out the hinges configuration; in other cases the location of plastic hinges has to be determined by using F.E. models.

The application of the principle of virtual work to the original un-deformed bridge shape is based on the following equation:

$$\alpha \sum_i P_i \cdot \delta_{x,i} - \sum_i P_i \cdot \delta_{y,i} - \sum_j F_j \cdot \delta_j = 0 \quad (3.1)$$

where α is the horizontal load multiplier that activates the mechanism, P_i represents the weight force of the arch voussoirs and of the filling voussoirs, F_j refers to the generic external force applied to the structure, δ stands for the virtual displacement of the load application point. The collapse limit acceleration a_0^* is derived as:

$$a_0^* = \frac{\alpha g}{e} \quad (3.2)$$

where e^* is the participant mass factor and g is the gravitational acceleration.

A detailed description of the procedure can be found in (Clemente, 1998) or (da Porto et al., 2007); the incremental load scheme called “M3”(Clemente 1998), where each voussoir is subjected to an horizontal force proportional to the vertical load acting on it was chosen in this work.

F.E. analyses were used in addition to KM in multi-span bridge models (2Ssl_2.2 and MSSl_3.2 bridges), to calibrate the position of plastic hinges and to determine, through non linear static analysis, the ultimate capacity of the bridge in transverse direction.

In Table 4.1 the mechanical parameters adopted for masonry in the F.E. models are reported.

Table 4.1. Average ranges of the masonry mechanical properties adopted in the F.E. models

MECHANICAL PROPERTIES OF BRICK MASONRY		
Elastic modulus	E_m	4000MPa ÷ 6000MPa
Compressive strength	f_c	3MPa ÷ 6MPa
Tensile strength	f_t	0.1MPa ÷ 0.3MPa

5. COLLAPSE MECHANISMS FOR DIFFERENT CLASSES OF MASONRY ARCH BRIDGES

The seismic vulnerability of a certain class of masonry arch bridges depends on the expected mechanism, that will affect the structure at the collapse in longitudinal or transverse direction. A brief description of the collapse mechanisms considered for each class is reported below.

- **Single span bridges with squat abutments (SSsa_1.1):** single span masonry arch bridges are generally characterized by massive abutments, which in most cases can be schematized as an infinitely rigid constraint. The most vulnerable element in the longitudinal direction is the masonry arch itself, which can collapse when subjected to horizontal accelerations developing an antimetric collapse mechanism through the formation of three rigid voussoirs and four hinges (see Fig. 5), which are situated at points where the thrust line crosses the masonry of the arch ring (Clemente, 1998). In this work the mechanism is called “A-L” collapse mechanism (A-L=arch mechanism in longitudinal direction). The geometric parameters that can influence the value of the seismic acceleration needed to trigger the mechanism of the arch are: span length L , span rise f , arch thickness S .

In the transverse direction, because of the high inertia of the abutment wall, the most vulnerable element is represented by the spandrel wall, which can fail by out-of-plane rotation (SW-T mechanism, see below).

- **Single span bridges with high abutments (SSha_1.2):** in single span bridges, if the abutments are high ($h/L > 0.75$), the longitudinal mechanism can involve both the arch and the abutments, becoming a global mechanism (da Porto et al., 2007). In this case, for large displacements, also the contribution of the fallow earth thrust and of the active and passive earth thrust mobilized at the limit state, have to be taken into account. In this paper this mechanism is called arch-abutment global mechanism in

longitudinal direction (AA-L). In the transverse direction the spandrel wall out-of-plane collapse mechanism is also to be considered.

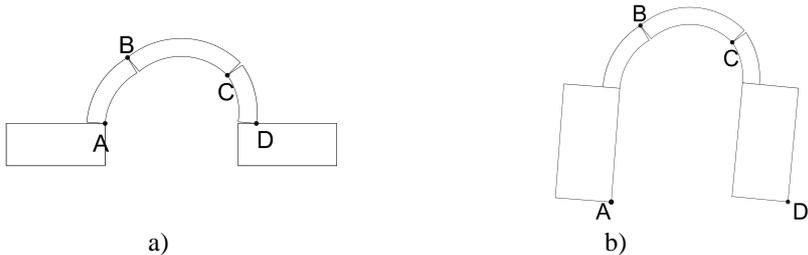


Figure 5. Single span bridge collapse mechanisms in longitudinal direction: a) collapse mechanism of the single arch with squat abutments (Bridge Class SSsa_1, A-L mechanism), b) collapse mechanism of the arch involving the high abutments (Bridge Class SSha_1, AA-L mechanism)

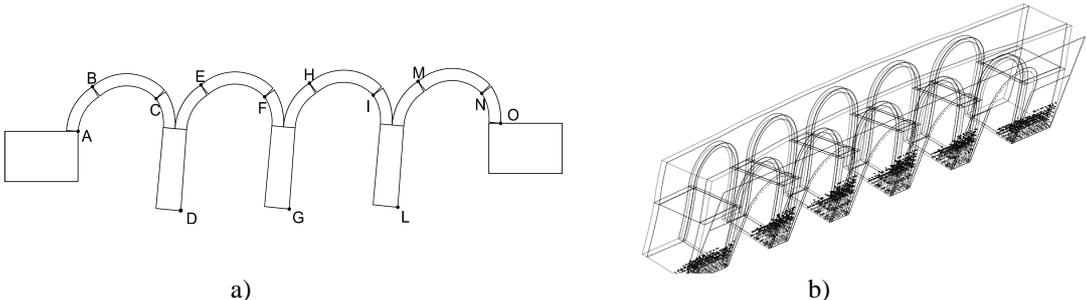


Figure 6. 2-3 Spans and Multi-Span Bridges with high abutment (2Ssl_2.2, MSsl_3.2): the arch-pier system can collapse in the longitudinal and transverse direction with the formation of plastic hinges at the pier bases.

- **Multi-span bridges with squat piers (2Ssp_2.1, MSsp_3.1):** for these classes the strong abutments continue to represent a fixed restraint for the arch, so each span can be regarded as independent, and the expected collapse mechanisms are the same as those considered for the single span bridge with squat abutments.

- **2-3 Spans and Multi-span bridges with high abutments (2Ssl_2.2, MSsl_3.2):** for these bridge classes the seismic vulnerability is affected by the slenderness of the piers, and influenced by the ratio H/B . In the longitudinal direction a global mechanism Arch-Piers (AP-L mechanism), with formation of plastic hinges at the pier bases, tends to develop at collapse (see also Clemente, 2010). In transverse direction, not only the local mechanism related to the out-of-plane rotation of the spandrel wall (SW-T mechanism) has to be considered, but also a global mechanism, involving both arch and piers (AP-T mechanism), which can only be identified with F.E. analysis. This mechanism is influenced by the transverse stiffness of the arched deck, which is connected to the abutments, and the restraint offered by the abutments is stronger in shorter bridges (belonging to class 2Ssl_2.2).

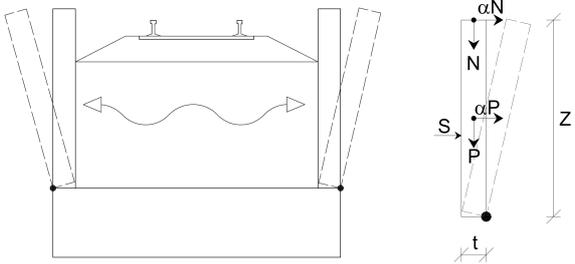


Figure 7. Out of plane overturning of the spandrel walls (SW-T mechanism)

- **All bridge classes (1.1-3.2): out of plane rotation of the spandrel wall.** The spandrel walls are subject to out of plane overturning (SW-T mechanism). This collapse is a local mechanism, and generally does not involve the structural safety of the arch, but it can compromise the support of the ballast and the rail tracks, and in the end the serviceability of the bridge.

6. PARAMETRICAL ANALYSIS

The parametrical analysis was carried out in order to calculate, for each bridge class (see Table 3.1), the correspondent horizontal load multiplier a_0^* activating the collapse mechanism.

Table 6.1. Ranges of the geometrical properties and type of the collapse mechanisms evaluated in the parametrical analysis (A-L= arch longitudinal mechanism; AA-L,= arch-abutment longitudinal mechanism; AP-L=arch-pier longitudinal mechanism ; AP-T=arch-pier transverse mechanism; SW-T=spandrel wall transverse mechanism)

BRIDGE CLASSES	SINGLE SPAN squat abutment (SSsq_1.1)	SINGLE SPAN high abutment (SSha_1.2)	2-3 SPANS slender piers (2Ssl_2.2), (MSsl_3.2)	MULTI-SPAN slender piers (2Ssl_2.2), (MSsl_3.2)
COLLAPSE MECHANISM	A-L SW-T	AA-L SW-T	AP-L AP-T SW-T	AP-L AP-T SW-T

Appropriate ranges of relevant geometrical factors such as span length, arch thickness, span-to-rise ratio, bridge width, pier height-section width ratio (H/B) were considered, and the corresponding limit accelerations for each relevant collapse mechanism were calculated through the application of kinematic method (KM) or F.E analysis.

Table 6.2. Single span bridges with high abutments (SSha_1.2): resistant acceleration values a_0^* for the mechanism AA-L, obtained with the kinematic method for $f/L=0.5$; $s'/L=0.1, 0.12$; $h/L=[0.60-1.8]$

L [m]	f/L	s'/L	h/L	a_0^* [g]	L [m]	f/L	s'/L	h/L	a_0^* [g]
3	0.50	0.10	0.60	0.292	3	0.50	0.12	0.60	0.298
3	0.50	0.10	0.75	0.259	3	0.50	0.12	0.75	0.263
3	0.50	0.10	0.90	0.226	3	0.50	0.12	0.90	0.229
3	0.50	0.10	1.20	0.185	3	0.50	0.12	1.20	0.187
3	0.50	0.10	1.80	0.137	3	0.50	0.12	1.80	0.138
4	0.50	0.10	0.60	0.276	4	0.50	0.12	0.60	0.281
4	0.50	0.10	0.75	0.246	4	0.50	0.12	0.75	0.250
4	0.50	0.10	0.90	0.215	4	0.50	0.12	0.90	0.219
4	0.50	0.10	1.20	0.177	4	0.50	0.12	1.20	0.180
4	0.50	0.10	1.80	0.132	4	0.50	0.12	1.80	0.134
5	0.50	0.10	0.60	0.276	5	0.50	0.12	0.60	0.281
5	0.50	0.10	0.75	0.244	5	0.50	0.12	0.75	0.248
5	0.50	0.10	0.90	0.212	5	0.50	0.12	0.90	0.215
5	0.50	0.10	1.20	0.173	5	0.50	0.12	1.20	0.175
5	0.50	0.10	1.80	0.127	5	0.50	0.12	1.80	0.129
6	0.50	0.10	0.60	0.275	6	0.50	0.12	0.60	0.279
6	0.50	0.10	0.75	0.243	6	0.50	0.12	0.75	0.247
6	0.50	0.10	0.90	0.211	6	0.50	0.12	0.90	0.214
6	0.50	0.10	1.20	0.173	6	0.50	0.12	1.20	0.175
6	0.50	0.10	1.80	0.127	6	0.50	0.12	1.80	0.128

The procedure followed to obtain the iso-acceleration curves in Figs. 9-11, is explained with reference to some results obtained for the “Single span bridges with high abutments” class (SSha_1.2), reported in Table 6.2: the limit acceleration value a_0^* is calculated with the kinematic method for a round arch ($f/L=0.5$), varying the value of abutment height-span length ratio (h/L), for two fixed values of the ratio s'/L (abutment thickness divided by span length).

From these data two interpolating lines were derived (see Figure 8), which represent the function $a_0^*=f(h/L)$ for a fixed value s'/L . Through the construction of a sufficient number of interpolating lines, it was possible to plot the iso-acceleration curves in Figure 9b.

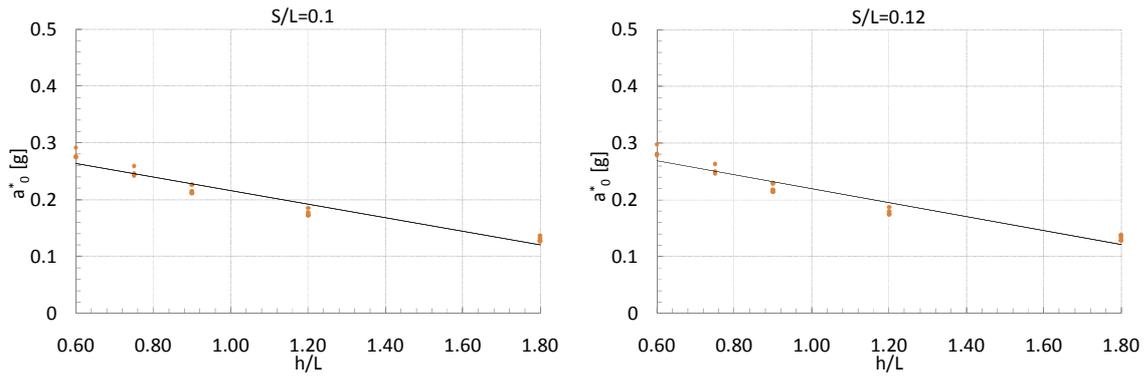


Figure 8. Interpolating lines obtained from the data of Table 6.2

In the following graphs (Figs. 9-11) the limit iso-acceleration curves, obtained with the same procedure described for Fig. 13b, are plotted as a function of the main geometric parameters, whose variation influences the ultimate capacity of the masonry bridge. The graphs are related to 5 of the collapse mechanisms reported in Table 6.1 for the different classes of masonry bridges.

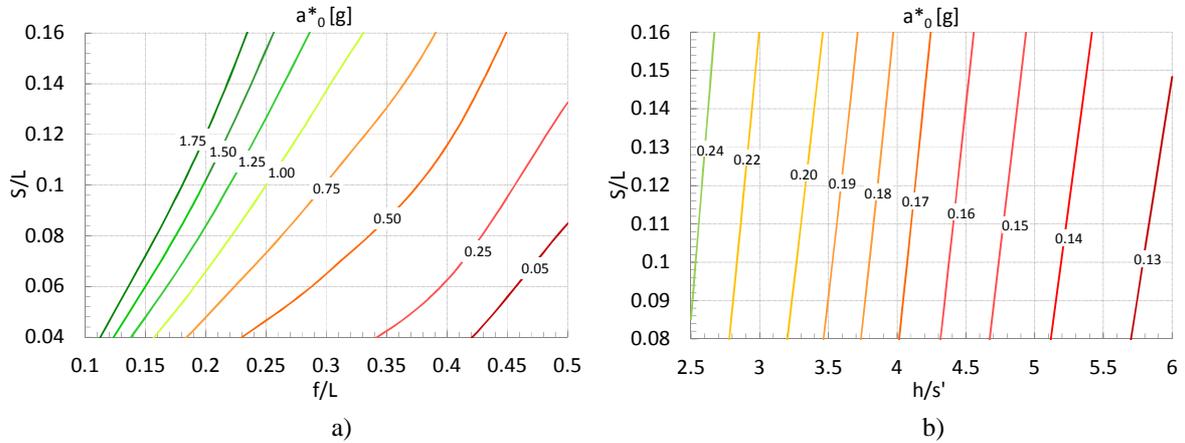


Figure 9. Single span bridges: a) Single span bridge with squat abutments SSsa_1.1, iso-acceleration curves for in-plane arch mechanism (A-L mechanism). b) Single span with high abutments SSsha_1.2), iso-acceleration curves for in-plane arch-abutment mechanism (AA-L mechanism)

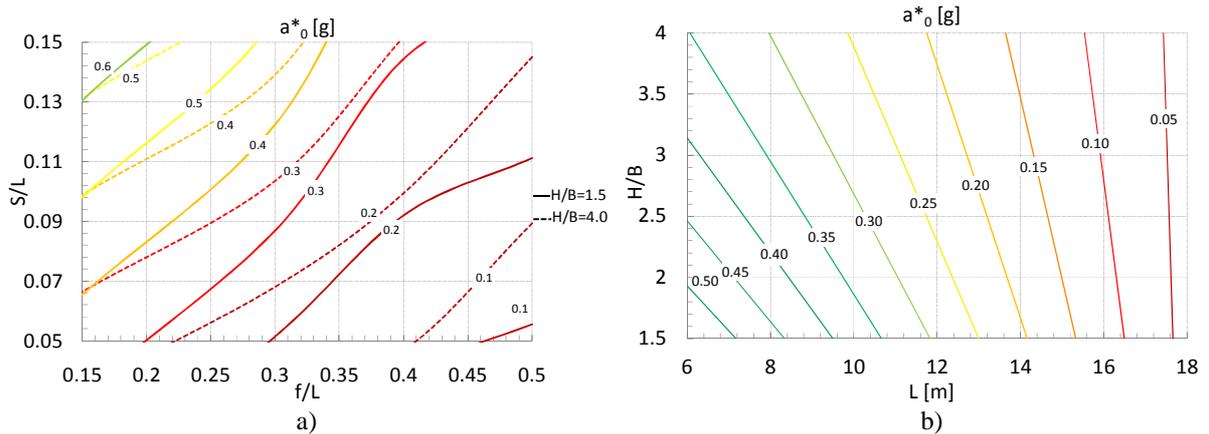


Figure 10. Multiple span bridges with slender piers (MSsl_3.2) bridges: iso-acceleration curves for a) in-plane arch-piers mechanism (AP-L mechanism), and b) out-of-plane global arch-piers mechanism (AP-T mechanism).

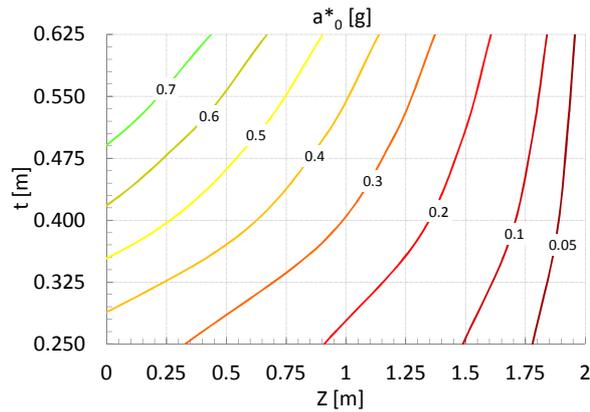


Figure 11. Iso-acceleration curves for out-of-plane spandrel wall rotation (SW-T mechanism)

8. CONCLUSIONS

Based on a statistical survey of an existing stock of 380 masonry arch bridges, a hierarchical classification of masonry bridges is proposed, considering the typological characteristics of the bridges and the type of expected collapse mechanisms under seismic action, is herein proposed. The limit ground acceleration corresponding to the ultimate capacity for different collapse mechanisms was calculated through a parametrical analysis for each bridge class, using the kinematic method and F.E. analyses. A series of parametric curves were obtained for the different bridge classes, representing the horizontal limit acceleration a_0^* as a function of a few geometric parameters, easily detectable by on-site inspections.

The iso-acceleration curves obtained can be used by technicians involved in the seismic safety check of existing railway masonry bridges, and can be easily implemented in the framework of a Bridge Management System for prioritizing further investigations and retrofitting interventions.

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