Displacement-based methods for the seismic assessment of masonry arch bridges

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ABSTRACT: Masonry arch bridges, built in the XIX century, are strategic manufacts in the Italian railways. In this paper, a methodology for the evaluation of the safety level of masonry bridges, in relation to the seismic action, is proposed. With this aim, the *performance-based approach*, recently proposed, in case of existing ordinary buildings, in several codes (ATC40, FEMA 273 and 356, new Italian Seismic Code), is applied in relation to these structures. This methodology, based on performance criteria, is feasible only through the use of a synthetic structural model, able to adequately simulate this particular typology, and useful both for push-over and dynamic analyses. In this way, the static analyses are validated through the results obtained by the dynamic simulations. The synthetic structural model is able to represent the bridge through a 3D description (using mono-dimensional non-linear elements, through which no tensile behaviour, limited strength in compression and shear damage due to sliding in the mortar joints are considered).

1 INTRODUCTION

Even if masonry arch bridges are not contemporarily designed structures, they represent the most frequent typology in the Italian railway network and they perform as strategic elements, even if of ordinary importance.

The Italian railway network was built during about a century, from 1830 to 1930. This peculiarity allows the identification of recurrent constructive technologies and bridge geometry. If one excludes those structures having unique features, due to specific building requirements, a few bridge typologies may be recognized. So, analyses can be finalized to bridges having recurrent geometric characteristics and they may well represent a wide population of structures.

2 STATISTICAL SURVEY

The North-Western part of the railway network was the object of a statistical survey, carried out in the archives of the National Railway Authority (Brencich *et al.* 2001, Gambarotta *et al.* 2002). This area has been selected because it presents a large variety of masonry bridges due to the geomorphology of the territory, where both mountain and plain railway lines are present. All kind of single- and multi-span bridge has been found, including a significant number of viaducts and some of the most endeavouring bridges. Therefore, the statistics are obtained on this sample area, but a spot analysis of the rest of the national territory has later shown that the information collected can be extended to the national bridge stock with a non significant error.

The Italian railway system presents a large number of single-span bridges, 1/3 of the total, whilst another 1/3 is represented by viaduct with more than three spans. It is worthwhile noting that the viaducts represent almost 1/3 of the whole Italian bridge stock. The number of 2- and 3-

spans bridges is unexpectedly low if compared to what happens in other European countries such as Britain.

Regarding the geometrical features of Italian railway arch bridges (Figure 1), it can be noted that the span ranges between 8 and 18m, but the typical span seems to be around 15 m, a value which has been recorded for many viaducts.

Almost all the arches are from 0.5 to 1.1 m thick in crown, generally from 5 to 7% of the span length and are of constant thickness; this is an important feature if the vault behaviour is analysed. Moreover, the bridge stock is made up of deep arches (33%), while approximately 40% of the arches are shallow

with rise-to-span ratio around 0.2.

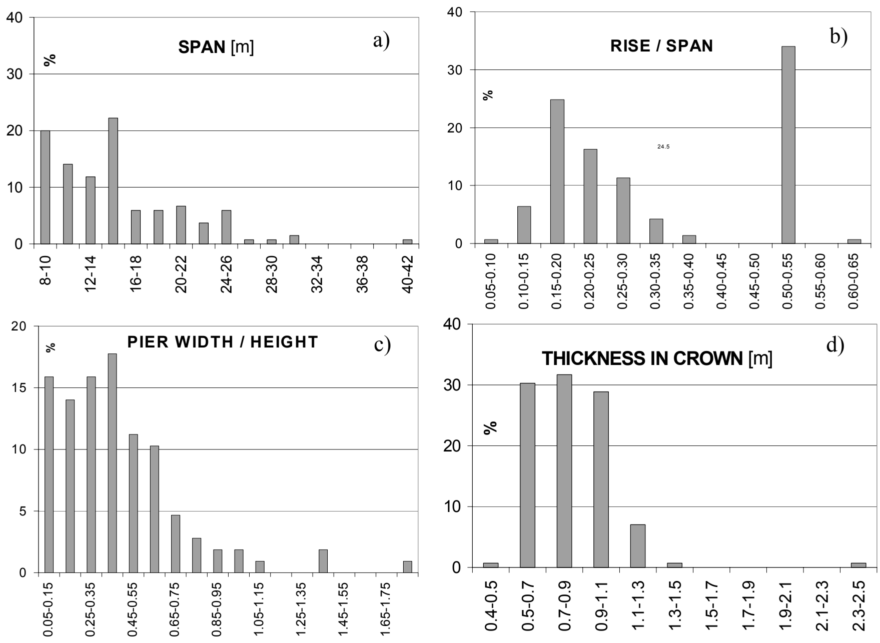


Figure 1: Statistical distributions: a) span length; b) rise/span ratio of the arch; c) pier width/height ratio; d) arch thickness in crown (Gambarotta *et al.* 2002).

Italian Bridges are generally very well backfilled; in fact, the backfill height is some 10 to 30% less than the rise. The springing width is 0.25-0.33 times the span.

No statistical rule can be found in the pier width that can vary from 10 to 45% of its height.

These data are not enough to define some prototype bridge models, but nevertheless allow some typical structural arrangements to be thought of as more frequent than others. As an example, 2- and 3-span bridges represent a minor part of the national stock, while single span (36.0%) and more than 3-span bridges (36.9%) are the most recurrent typologies. This fact can be explained considering that most of the Italian bridges had been built in a rather short period of time, 50 years only, following the French school, i.e. importing knowledge from a country that had already developed a high level technology for masonry bridges. In this way, it is not likely that significant changes in the building techniques could have been introduced in such a short period.

3 SEISMIC CONCERNS ABOUT MASONRY ARCH BRIDGES

Seismic history of railway arch bridges is relatively short; as a consequence, scarce information has been found about damage due to earthquakes and this situation leads to problematic issues in order to study the seismic response of these structures.

Masonry bridges do not seem to be particularly susceptible to damage in case of low-severity earthquakes, unless in presence of strong structural deficiencies. Nevertheless, destructive events may reasonably cause damage to various constitutive elements of the bridge: the investigation about this topic seems to be very interesting, also in relation with the territorial location of the railway lines.

3.1 Seismic hazard and damage to bridges

The Italian railway system is comprehensive of main and secondary lines, along about 16108 km. From data of the National Railway Authority archives, about 55000 bridges are present on the lines; 9000 of those are estimated to be masonry arch bridges of significant span.

From the national catalogue NT4.1 (GNDT – National Group for the Defense against Earthquakes), maps concerning the epicentral intensity (degrees IX, X, XI of the Mercalli scale), in Figure 2-a, and the related macroseismic magnitudes (Figure 2-b) were evaluated in case of events from 1830 up to now.

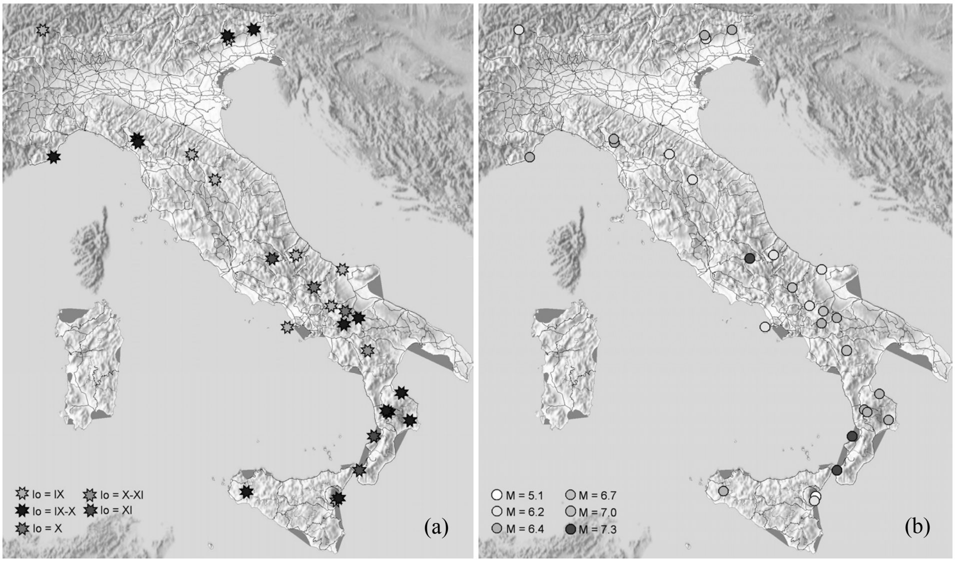


Figure 2: Location of recent seismic events (from 1830 up to now): (a) epicentral intensity equal and higher than IX degree; (b) magnitude related to these events. In black, the railway lines.

From the previous maps, several railway lines in high seismicity areas may be identified: nevertheless, seismic damage to arch bridge is not well documented.

In Italy, the seismic events are frequent, but not extremely severe. Even if the epicentral area of many earthquakes is located near the railways, masonry bridges might perhaps be undamaged because if the weak ground motion. On the other hand, especially in case of ancient events, the low exposure of the zones in which bridges might be built (mountain or not very populated areas) can led to lack of information even in case of strong earthquakes.

Moreover, even if some damage to bridges is mentioned, the documentation is hardly available, because, during the 18th and 19th centuries, the National Railway Authority was subjected to many changes and part of the archives got lost.

4 STRUCTURAL MODEL OF THE BRIDGE

The 3D model is based on the spatial description of the behaviour of the masonry bridge elements. It is necessary to understand and properly simulate the dynamic and structural properties of the bridge constitutive elements. A mono-dimensional non-linear element, having truncatedpyramid shape, is used. This element is divided into three portions. In the central part, the shear behaviour (with non-linearity, damage and sliding related to frictional effects – in a pre-chosen direction) and the torsional behaviour develop. In the two layers, inferior and superior, the bending and axial effects (considering no-tension behaviour and crushing) are modeled. The reader may find the detailed formulation in Resemini (2003). These elements are derived from the macroelements already proposed for the analysis of masonry walls (Gambarotta & Lagomarsino 1996, Penna 2001).

The mesh and element assembly procedures arise from structural and typological investigations.

4.1 Constitutive elements of the masonry bridge and their structural behaviour

The constitutive portions of a masonry bridge (Albenga 1958), in Figure 3, can be briefly identified with: the *vault* (or *arch*) that is the structure supporting the pavement and super-structures; the supporting structures (*pier*, *abutments*) of the vaults; the *foundations*; the apparently non-structural elements, such as *backfill*, *fill* and *impermeable coat,* built above the vaults to create a plane pavement. The fill is sided by two *spandrel walls*, which are built on the external parts of the barrel vault.

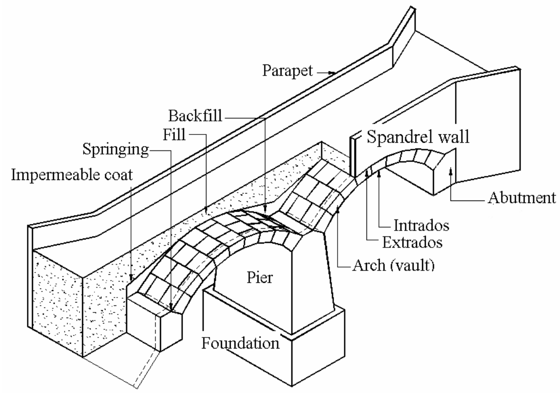


Figure 3: Axonometric section plane of a masonry bridge.

The vaulted element is thought so that the radial sections may work prevalently under compressive loading; moreover the behaviour is approximately two-dimensional (the resistant mechanism mainly develops in the longitudinal plane), excluding load redistribution due to the different stiffness of the spandrel walls and the central portion of the vault. Under dead load and vertical load condition, this hypothesis seems to be adequate. This is no more verified in case of seismic event, because of which relevant transversal effects could develop and the vault could behave in a three-dimensional way. The model have to be capable to simulate both the damage mechanisms in the thickness of the arch (Figure 4-a), including crack opening and crushing due to compressive forces, and those involving the whole vault (Figure 4-b), connected to crack opening, crushing due to compressive forces, shear forces and sliding in the mortar joints.

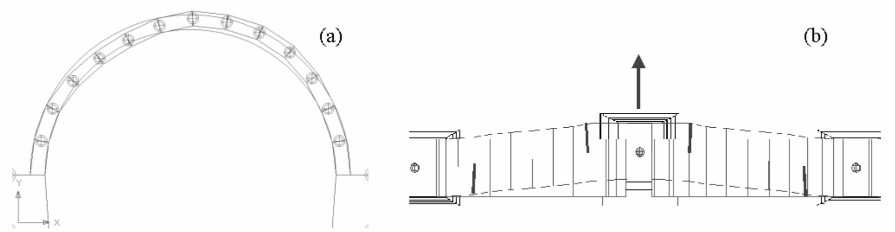


Figure 4: Damage mechanism schematisation: (a) in the arch thickness; (b) in the whole vault (top view).

In the piers, the damage mechanisms connected to the seismic action include crack opening, crushing due to compressive forces, shear forces and sliding in the mortar joints in the transversal direction (along which the pier is generally squatter). The mesh and element assembly procedures (Figure 5) arise from these crucial structural and dynamic remarks.

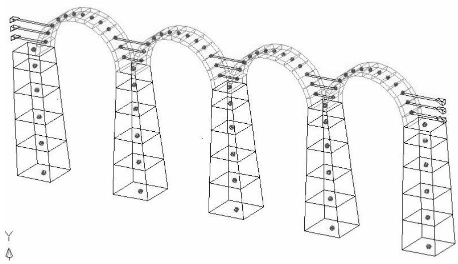


Figure 5: Masonry bridge mesh through mono-dimensional non-linear elements.

By using the mono-dimensional non-linear element (having two nodes), the masonry bridge can be meshed in order to better simulate the global structural behaviour. The vault elements are assembled along the circular direction of the vault itself. In the piers and abutments, the elements are arranged along the vertical direction and the pre-defined damage direction, in case of shear and sliding, is the transversal one. The model may account for the spandrel walls and the backfill.

5 SEISMIC RESPONSE EVALUATION OF MASONRY BRIDGES

Aiming to verify the effectiveness of the structural model, non linear-simulations (dynamic and pushover analyses) were performed on typical configuration of Italian railway bridges. A *displacement-based approach*, recently proposed, in case of existing ordinary buildings, in several codes (ATC40, FEMA 273 and 356, new Italian Seismic Code), is applied in relation to masonry arch bridges. This methodology, based on performance criteria, is feasible only through the use of a synthetic structural model, able to simulate adequately these particular masonry structures, and useful both for pushover and dynamic analyses. In this way, the static analyses are validated through the results obtained by the dynamic simulations.

In order to perform non-linear seismic analyses of arch bridges, various procedures has been implemented (Galasco *et al.* 2004): e.g. 3D pushover analysis with fixed load pattern and 3D time-history dynamic analysis (Newmark integration method; Rayleigh viscous damping), considering uniform or spatially varying motion. The pushover procedure, with an effective algorithm, transforms the problem of pushing a structure maintaining constant ratios between the applied forces into an equivalent incremental static analysis with one d.o.f. displacement response control.

In the following, two case studies are proposed: the structures were selected among the bridges on the Italian railways having typical geometries and constructive features. The availability of experimental data and geotechnical information was considered as discriminating factor.

5.1 Displacement-based procedures for the seismic analysis

With regard to the representation of the seismic demand, some specific considerations are needed for the response spectra reduction in the particular case of masonry arch bridges. Because of the nearly non-tensile strength of masonry that cause widespread cracking, the structural behaviour is strongly non-linear, even in the low-displacement range. Moreover, the equivalent single deggre of freedom (SDOF) system representative of the bridge is not of obvious definition, because of the complex dynamic interaction between the vault system and the piers. These considerations imply to check the feasibility of the conventional procedures, in order to be applied to masonry arch bridges.

One of the most used procedures is the inelastic spectrum, in which the reduction factors are function of the ductility (Fajfar 2000). However, when inelastic spectra are employed, the resulting seismic demand in terms of displacement is strongly influenced by the definition of the ductility μ that is difficult to be determined for masonry structure, in which also the initial range of the pushover curve is affected by some non-linearity.

As a feasible alternative, the use elastic demand spectrum considering overdamping may be proposed. Even if some issues were discussed in case of rocking elements belonging to monumental structures (Resemini *et al.* 2006), masonry arch bridges belong to a particular typology, for which deeper studies have to be developed. This research is now in progress by the authors, but only preliminary results are up to now available. So, the following examples, carried out using the inelastic spectra, and other case studies were yet proposed elsewhere (Resemini 2003).

In order to represent the seismic demand, reference was made to the Curves of Inelastic Demand (Lagomarsino *et al.* 2002). The Eurocode 8 elastic spectra (5% damping) are converted into inelastic spectra with constant ductility, using simplified relationship in terms of ductility μ and reduction factor (Fajfar, 2000).

Being T the elastic period corresponding to the limit-elastic displacement d_y of the equivalent SDOF system, the displacement demand spectrum S_d may be represented as in eq.(1).

$$\begin{cases} S_d = 2.5 a_g \dfrac{T_C}{2\pi}\sqrt{\dfrac{d_y}{S_a}} - \left(\dfrac{T_C}{2\pi}\sqrt{\dfrac{S_a}{d_y}} - 1\right) d_y & S_a > \dfrac{4\pi^2}{T_C^2} d_y \\ S_d = 2.5 a_g \dfrac{T_C}{2\pi}\sqrt{\dfrac{d_y}{S_a}} & S_a \le \dfrac{4\pi^2}{T_C^2} d_y \end{cases} \tag{1}$$

where a_g is the peak round acceleration, T_C the spectral period at the end of the constant acceleration range

If we consider a structural system having fixed d_y, eq.(1) represents the displacement demand, for which the spectral reduction factor is implicitly accounted for. The intersection between the Curve of Inelastic Demand and the capacity curve of the bridge lead to the expected performance, without using iterative procedures. As previously highlighted, the evaluation of the limit-elastic displacement d_y is crucial for the method.

In this case, the displacement in the capacity curve of the equivalent SDOF system $S_{d,\mathrm{eq}}$ is evaluated through a linear interpolation between the horizontal displacements of two control points at the same height, chosen as the height of the resulting applied force in the pushover analysis.

The base-shear force V is converted into spectral acceleration in the capacity curve of the equivalent SDOF system, evaluating the fundamental period T_1 of the structure and dividing by the effective mass m_{eq} as in eq.(2).

$$m_{eq} = \frac{V^e T_1^2}{4\pi^2 S_{d,eq}^e}, \tag{2}$$

where the base-shear force V^e and the equivalent displacement $S_{d,eq}^e$ are evaluated in the initial branch of the pushover curve.

This approach is applied both to the longitudinal and transversal analyses of the bridge, referring to orthogonal components of the seismic action.

5.1.1 Seismic analysis of the Cantalupo Viaduct (longitudinal component)

In order to give a first glance on the procedure, an example will be discussed referring to the parts of the procedure already developed and tested. Table 1 summarizes the main geometrical parameters of the Cantalupo viaduct.

Table 1. Main features of the Cantalupo viaduct.

VAULT – 6-span bridge			
Geometric arch span (intrados) s [m]	18.50	Width [m]	5.40
Geometric rise (intrados) r [m]	9.25	Geometric r/s	1 / 2
Structural arch span (intrados) s_{str} [m]	14.75	Structural r_{str}/s_{str}	1 / 4
Structural arch rise (intrados) r_{str} [m]	3.67	d/s_{str}	1/13.8
Arch thickness d [m]	1.07	f/d	0.7
Fill in crown f [m]	0.75	Structural arch skewbacks at	37.12°
PIER – with varying section			
Min-Max height h_{min} - h_{max} [m]	10.68-31.05	Pier slope p [%]	3.5

The Cantalupo viaduct (Figure 6) is a 6 span in-service railway bridge at km 13.549 on the Genoa-Ovada-Aqui Terme line in North-Western Italy.The vault are made of solid clay brickwork, while spandrels of rough stone masonry (serpentine); the fill consists of gravel. Piers are made of the same masonry as the spandrels, have 3.00x3.80 m at the arch skewbacks and enlarge toward the base with a 3.5% longitudinal and 4% transversal slope. Experimental testing provides the vibration periods and modal shapes; from those data, the elastic mechanical characteristic of the masonry were derived.

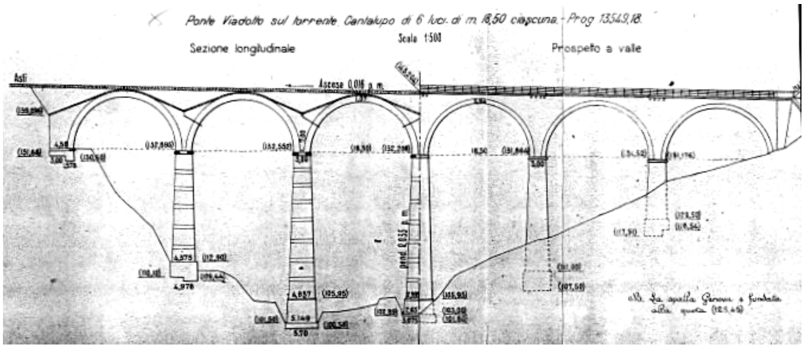


Figure 6: Cantalupo viaduct on the railway line Genoa-Ovada-Acqui Terme (from the National Railway Authority archive – Genoa).

The simulation involves the longitudinal response of the bridge. The structural model of the Cantalupo viaduct accounts for the spandrel walls and the backfill as geometry suggests.

Pushover analyses were performed, controlling the displacement of a significant point and applying different load-distribution conditions: (a) the 1st modal shape; (b) uniform; (c) triangular for the piers and uniform for the vaults.

The performance point evaluation through the pushover analyses and the inelastic spectra is validated through the dynamic results in terms of maximum equivalent displacement, using acceleration time-histories matching the Eurocode 8 spectra (peak ground acceleration PGA equal to 0.1g and 0.2 g), as in Figure 7.

It is worth noting that the static method using inelastic spectra well predict the dynamic response in terms of displacement. The obtained values are on the safe side, especially in case of low PGA, beyond which, in this case, the structural non-linearity becomes stronger. Moreover, different force distributions do not provide significant scatter of the results (both for the pushover and dynamic simulations).

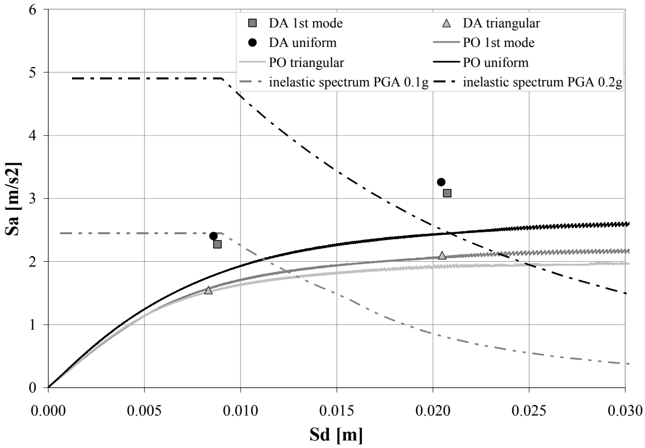


Figure 7: Cantalupo viaduct: comparison between static and dynamic procedures.

It should be highlighted that the main damage feature is related to the cracking: shear damage is not pointed out, while compressive failure are limited to masonry portion in the arch.

5.1.2 Seismic analysis of the Cevetta bridge (transversal component)

Cevetta bridge was analysed in order to get information about the seismic response in case of transversal excitation. Table 2 summarizes the main geometrical parameters of the bridge model.

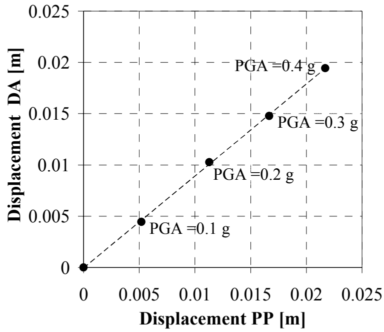
Table 2. Main features of the Cevetta bridge.

VAULT – 3-span bridge			
Geometric arch span (intrados) s [m]	17	Width [m]	6
Geometric rise (intrados) r [m]	8.5	Geometric r/s	1 / 2
Structural arch span (intrados) s_{str} [m]	11.2	Structural r_{str}/s_{str}	0.19
Structural arch rise (intrados) r_{str} [m]	2.11	d/s_{str}	1/11.2
Arch thickness d [m]	1	f/d	1.8
Fill in crown f [m]	1.8	Structural arch skewbacks at	49°
PIER – with varying section			
Min-Max height h_{min} - h_{max} [m]	13.2	Pier slope p [%]	4

The structural model of the Cevetta bridge (Figure 8-a) accounts for the spandrel walls and the backfill as geometry suggests. By analogy with the previous example, the three force distributions were assigned; obviously the 1st mode distribution refers to the transversal modal shape, in which the pier vibration is in-phase.

Step-by-step dynamic simulations were performed using acceleration time-histories matching the Eurocode 8 spectra (PGA from 0.1g to 0.4 g). The correlation between the performance-point displacement (PP) and the maximum dynamic response (DA) is reported in Figure 8-b.

(a)



(b)

Figure 8: (a) Cevetta bridge; (b) correlation between the performance-point displacement (PP) and the maximum dynamic response (DA) - 1st mode force distribution.

The static method with inelastic spectra pointed out slightly overestimated results: this trend is constant increasing the PGA value of the input.

In this case study, the results in terms of capacity (pushover curve) are strongly affected by the choice of the force distribution: the approximation of the dynamic results for the performance point evaluation is very good using the 1st mode force distribution, but not in the other conditions.

In fact only in that case, a sudden strength fall, due to damage in various elements of the bridge, is noticed; the resisting system is modified, but the structure is able to increase the base-shear force, as the displacement increases.

6 FINAL REMARKS

Even if, up to now, seismic vulnerability of masonry arch bridges was not systematically studied, from the results of the numerical simulations (pushover and dynamic) using the proposed structural model, it can be achieved that those structures seem to well behave against earthquake. Damage occurrence was noted in case of severe action.

The case studies, briefly described in order to set out the response-prediction capability of the proposed method, highlight a good correspondence between pushover and dynamic results. In the analyses, the main damage feature is related to cracking and rotation both in the vault elements and in the piers. The compressive stress is not particularly high in respect to the masonry strength (generally, good quality material and technology were employed). The bridge may rely on quite high ductility, especially in case of high piers, but also in relation to the displacement capacity of the whole arch-pier system.

On the other hand, the results pointed out that the choice of the acceleration time-histories affects the structural response. In order to overcome this problem, non-linear dynamic analyses on the equivalent SDOF systems will be performed employing a wider set of input ground motions: recorded accelerograms, selected to have the average spectrum matching the Eurocode 8 one, will be considered (e.g. Iervolino *et al.* 2005).

Moreover, analyses involving the overdamped spectra should be needed in order to study the feasibility of this procedure in case of masonry bridges. At the present time, cyclic pushover simulations are in progress, endeavouring to better understand the hysteretic dissipation behaviour in this kind of structures. This is aimed to the correct definition of the damping relationship in the structural model.

The influence of high-frequency modal shapes for the force-distribution choice and the effect of the loose fill may be interesting topics. Their effects on the dynamic response and the related prediction of displacement capacity need to be investigated.

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