SEISMIC ASSESSMENT OF MASONRY ARCH BRIDGES AND PARAMETRIC INVESTIGATION OF STRUCTURAL REINFORCEMENT

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SUMMARY

The evaluation of the load carrying capacity and the rehabilitation of existing masonry arch bridges represent a crucial task, related with the human safety and the conservation of architectural heritage. Considering the large number of masonry arch bridges still in service along the existing road network, the assessment of their seismic vulnerability is of relevant importance. In this paper a numerical procedure based on the limit analysis for the assessment of masonry arch bridges subjected to permanent and seismic loads is presented. A parametric investigation on the effect of the backfill internal friction angle and abutments height is performed. Moreover, the effect of two types of strengthening interventions, namely the arch thickening and the application of fiber composite materials, are furthermore discussed.

Keywords: *Arch bridge, masonry, earthquake, strengthening.*

1. INTRODUCTION

The evaluation of the load carrying capacity and the rehabilitation of existing masonry arch bridges represent a crucial task, related with the human safety and the conservation of architectural heritage [1]. Effectively, the respect of the new structural codes often requires specific strengthening measures in order to make the structure capable to withstand to external live loads. In this paper a numerical procedure for the vulnerability assessment of masonry arch bridges is presented. The method, leading to the determination of the seismic load multiplier and the corresponding collapse mechanism, is based on the limit analysis and refers to Heyman's hypotheses. Thus a no-tensile material, infinite compressive strength and a pure rotational failure mechanism at the joints are assumed [2]. The arch is considered subjected to the self-weight and to the inertial loads induced by the seismic action. In the calculus, the load effect due to the presence of the backfill is taken into account. This paper is thus aimed at studying the influence of the backfill and the soil pressure on the collapse behaviour of the masonry arch bridge and analyzing the effect of the interventions on the seismic load multiplier and on the collapse mechanism. Moreover, two types of measures are investigated: the arch thickening and the strengthening of the arch by means of composite materials.

2. ROLE OF THE INFILL IN THE SEISMIC LOAD BEARING CAPACITY

The essential role played by the infill in the stability of arched structures subjected to vertical loads is well known since the medieval age. Whereas the stabilizing effect of its self-weight has always been well know, only in the last decades the advantages of its stiffness have started to be taken into account. The effect of the infill self-weight is well documented by Gago et al. [3] who show that loading distributions similar to that of the infill self-weight in circular arches produce quasi circumferential lines of thrust. The Authors also highlighted the following two favourable effects of the infill: 1) a restriction of the lateral movement of the loaded voussoirs, giving rise to a smaller effective span of the arched structure, and 2) a distribution over a wider length of the arch of any concentrated load applied to the top of the infill.

Molins and Roca [4] were among the first to take into account the infill stiffness, by means of its discretization into a system of equivalent linear elements. Their analysis showed that only when the active contribution of the spandrel infill was included, satisfactory agreement could be obtained between the numerical simulation and the corresponding experimental measurements. More recently, the effect of the infill has been modelled by means of lateral springs [5], horizontal pressures just behind the abutments or directly by using a finite element mesh.

As far as the seismic action is concerned, several studies exist for isolated masonry arches [6, 7], while other studies assume the presence of the infill but applying the seismic action only to the external live load [8]. Although in other papers the effect of lateral active and passive soil pressure has been included [9, 10], the role of the infill in the in-plane seismic load bearing capacity of masonry arches is still not well defined.

3. PROPOSED METHOD

3.1. Geometrical description and loading condition

The failure condition of a masonry arch bridge in presence of its self-weight, the weight of the backfill and the seismic action has been investigated by referring to limit analysis [2]. Three conditions are assumed to be verified at the collapse: *i)* resistance criterion, *ii)* equilibrium and *iii)* mechanism condition. The first and second one correspond to the existence of a line of thrust everywhere contained inside the boundary of the arch thickness and satisfying the equilibrium with the acting loads. The third condition requires the activation of a four-hinges mechanism. The following Heyman hypotheses are assumed for the masonry: no-tensile strength, infinite compressive strength and no-sliding failure condition between the voussoirs. An iterative procedure, briefly described below, has been used to determine the collapse mechanism and the corresponding seismic load multiplier.

The masonry arch bridge has been analysed by considering the presence of the circular arch, the abutments and the backfill. Since the latter is taken into account uniquely as a load, the bearing structure is composed of the arch and the abutments. The analysis was carried out by considering a 2D problem. The following parameters, shown in Fig. 1, define the geometry of the structure: the span length l, the rise f and the thickness s of the arch, the height h and the width b of the abutments, the height h_c of the backfill at the crown. The out-of-plane depth is denoted by d.

The arch and the abutments are discretized into n voussoirs, numbered from left to right. The resulting $n+1$ joints are obtained for the arch and abutments by radial and horizontal cuts respectively. After the definition of the geometrical parameters, the coordinates of the following points are determined by referring to the Cartesian reference system (z, y) of Fig. 2: centre of gravity G_i of the ith voussoir, intrados I_j, extrados S_j and geometric centres P_j of the jth joint. The backfill is discretized into r elements obtained by intersecting the horizontal top line and the extrados with vertical lines starting from the points S_j, as shown in Fig. 1. In this way the horizontal size of the backfill is $l + 2\,b$. The coordinates of the centre of gravity $G_{b,k}$ of the kth backfill elements are then evaluated (Fig. 2).

The structure is subjected to the action of the self-weight of the voussoirs F_i, the backfill elements weight $F_{b,k}$, the active S_a and passive S_p soil pressure acting on the arch and abutments, the seismic increment of the active soil pressure P_{aE} - S_a (Fig. 3) and the seismic actions related to the mass of the voussoirs $F_{i,S}$ and backfill $F_{b,k,S}$. The resultant loading system consists of vertical and horizontal loads applied at the centres of gravity of the voussoirs and backfill (Fig. 2).

The self-weights of the ith voussoir and kth backfill element, applied at the corresponding centres of gravity G_i and $G_{b,k}$, can be evaluated as follows:

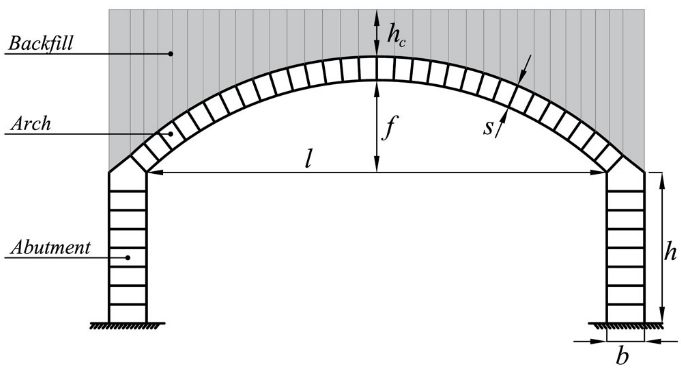


Fig. 1. *Nomenclature and geometry of the masonry arch bridge.*

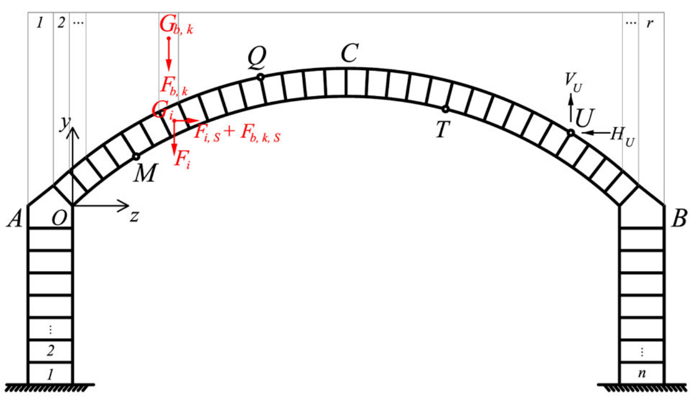


Fig. 2. *Weight of the voussoirs and backfill and inertial forces acting on the four-hinges arch.*

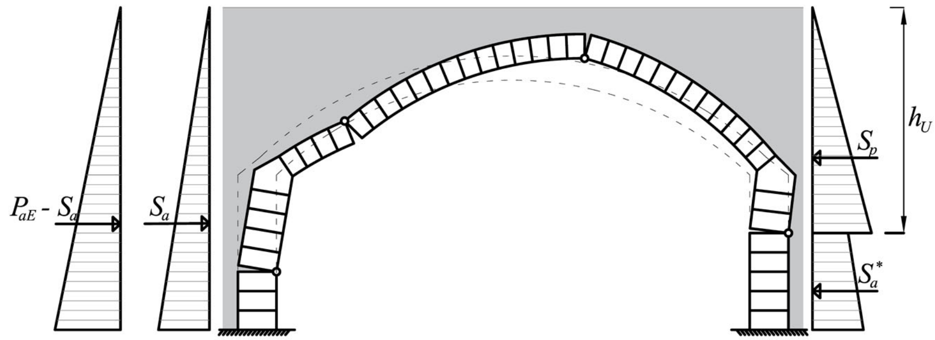


Fig. 3. *Active, passive and seismic infill pressure acting on the arch bridge.*

$$F_i = \gamma_m \cdot A_i \cdot d \tag{1}$$

$$F_{b,k} = \gamma_b \cdot A_{b,k} \cdot d \tag{2}$$

where γ_m and γ_b represent the masonry and backfill specific weight respectively, A_i the area of the ith voussoir, $A_{b,k}$ the area of the kth backfill element, with i = 1 to n and k = 1 to r. The seismic action is defined by a system of horizontal forces proportional to the vertical weights of the voussoirs and the backfill through a multiplier μ; they are directed, without loss of generality, from left to right. The seismic force related to the ith element of the arch or abutments is applied at the centre of gravity G_i:

$$F_{i,\mathrm{S}} = \mu \cdot F_i \tag{3}$$

with i = 1 to n. Based on the contents of the next paragraph, the seismic force associated with the mass of the backfill has been determined by considering only the masses placed on the left side of the bridge, thus resulting:

$$F_{b,k,\mathrm{S}} = \mu \cdot F_{b,k} \tag{4}$$

with k = 1 to r/2, being each force applied at the centre of gravity of the underneath voussoir (Fig. 2).

3.2. Infill horizontal pressure

In the evaluation of the seismic load multiplier μ, the pressure of the infill on the arch and the abutments cannot be neglected. However, its correct evaluation is not simple due to the impossibility of knowing, with the necessary accuracy, the geotechnical and mechanical properties of the soil, sometimes non-homogeneous, employed for the construction of the backfill.

In the present paper, the active horizontal pressure of the soil has been evaluated according to the Rankine theory, assuming a friction angle of the backfill ϕ', planar failure surfaces and level backslope of the infill. The total active thrust is thus equal to:

$$S_a = \frac{1}{2}\gamma_b\ k_a \cdot (h + f + s + h_c)^2 \cdot d \tag{5}$$

where k_a is the active pressure coefficient. A Rankine passive pressure has been applied to the portion of the arch that moves into the backfill, reduced according to suitable empirical relations [11, 12]:

$$S_p = \frac{1}{2}\gamma_b\, h_U^2\, k_{pr} \tag{6}$$

where $k_{pr} = m_p \cdot k_p$ being k_p the passive pressure coefficient, h_U the vertical distance between the hinge U and the top horizontal line of the backfill and m_p the reduction coefficient of the passive pressure. As suggested in [13] it has been assumed $m_p = 0{,}33$ in order to take into account the improbable mobilitation of the total passive pressure, since this would be verified only in presence of high displacements of the structure. The proposed method is, however, approximated since it ignores the stiffness and the resistance of the infill placed directly above the arch that opposes to the arch deformation. To balance this approximation the seismic mass of the infill material placed above the arch has been reduced: considering the assumed direction of the seismic loads, only the inertial contribution of the masses of backfill placed at the left side from the crown has been taken into account.

According to [14], masonry arch bridges subjected to in-plane seismic action can be considered as structures which essentially follow the horizontal seismic motion of the ground ("locked-in" structures). Thus, these structures do not experience significant amplification of the horizontal ground acceleration. Following the Mononobe-Okabe pseudo-static approach, the active thrust in seismic conditions has a resultant P_{aE} and includes the contribution of the static active pressure S_a:

$$P_{aE} = \frac{1}{2}\gamma_b\ k_{aE} \cdot (h + f + s + h_c)^2\ (1 - k_v) \cdot d \tag{7}$$

where k_{aE} is the Mononobe-Okabe active pressure coefficient, obtained with a friction angle between the abutments and the infill equal to $2/3\,\phi'$, and k_v is the vertical ground acceleration due to the earthquake. Both the active resultant thrust S_a and the seismic increment $P_{aE} - S_a$ are applied at $(h + f + s + h_c) / 3$ from the base of the abutments. The configuration of the adopted soil pressures is shown in Fig. 3.

3.3. Collapse mechanism

The collapse mechanism of the structure and the corresponding seismic load multiplier μ are attained by an iterative procedure [15, 16]. A first trial configuration of the hinges position has been assumed and the equilibrium imposed. If the resulting line of thrust is everywhere inside the masonry, the resistance criterion is satisfied and the solution is found. On the contrary, if the line of thrust falls outside the arch, the position of the hinges must be changed and the equilibrium imposed again. In order to get the right solution, each hinge is shifted toward the joint where the distance between the centre line of the arch and the line of thrust is maximum.

Let us denote by V_U and H_U the vertical and horizontal reactions at hinge *U* (Fig. 2), by *M*, *Q*, *T* and *U* the four hinges corresponding to the *m*, *q*, *t* and *u* joints and by R_i the resultant horizontal soil pressure acting on the ith voussoir, being its application point assumed at $y_{R,i}$. Considering that hinges *M* and *Q* take place at the left side of the structure, while *T* and *U* at the right one, the moment equilibrium about the remaining hinges *M*, *Q* and *T* gives:

$$\begin{cases} H_U(y_T - y_U) + V_U(z_T - z_U) - \sum_{i=1}^{n_{TU}} F_i(z_T - z_{G_i}) - \sum_{k=1}^{r_{\omega\xi}} F_{b,k}\left(z_T - z_{G_{b,k}}\right) + \sum_{i=1}^{n_{TU}} R_i(y_T - y_{R_i}) - \mu \cdot \sum_{i=1}^{n_{TU}} F_i(y_T - y_{G_i}) = 0 \\ H_U(y_Q - y_U) + V_U(z_Q - z_U) - \sum_{i=1}^{n_{QU}} F_i(z_Q - z_{G_i}) - \sum_{k=1}^{r_{\eta\xi}} F_{b,k}\left(z_Q - z_{G_{b,k}}\right) - \sum_{i=1}^{n_{QC}} R_i(y_Q - y_{R_i}) + \sum_{i=1}^{n_{CU}} R_i(y_Q - y_{R_i}) + \\ \qquad - \mu \cdot \sum_{i=1}^{n_{QU}} F_i(y_Q - y_{G_i}) - \mu \cdot \sum_{k=1}^{r_{\eta C}} F_{b,k}(y_Q - y_{G_k}) = 0 \\ H_U(y_M - y_U) + V_U(z_M - z_U) - \sum_{i=1}^{n_{MU}} F_i(z_M - z_{G_i}) - \sum_{k=1}^{r_{\zeta\xi}} F_{b,k}\left(z_M - z_{G_{b,k}}\right) - \sum_{i=1}^{n_{MC}} R_i(y_M - y_{R_i}) + \sum_{i=1}^{n_{CU}} R_i(y_M - y_{R_i}) + \\ \qquad - \mu \cdot \sum_{i=1}^{n_{MU}} F_i(y_M - y_{G_i}) - \mu \cdot \sum_{k=1}^{r_{\zeta C}} F_{b,k}(y_M - y_{G_k}) = 0 \end{cases} \tag{8}$$

where *r* and *n* refer respectively to the number of elements of backfill and structure. The associated subscripts identify the delimiting joints, being: *n*/2+1 the index joint at the crown *C*, ω = *T* or *B* if the hinge *T* is placed respectively on the arch or on the right abutment *B*, ξ = *U* or *B* if the hinge *U* is placed respectively on the arch or the right abutment *B*, η = *Q* or *A* if the hinge *Q* is placed respectively on the arch or on the left abutment *A*, ζ = *M* or *A* if the hinge *M* is placed respectively on the arch or the left abutment *A* (Fig. 3). The system of equations (8) can be solved in order to provide the reactions at hinge *U* and the seismic load multiplier μ. The complete knowing of the load system allows the determination of the eccentricity of the normal force at each joint and the drawing of the thrust line. If the resistance criterion is satisfied, namely the thrust line is at each joint inside the masonry, then the position of hinges identifies the actual failure mechanism and the corresponding seismic load multiplier. Otherwise, necessarily the hinges have to be moved and the procedure repeated.

4. VALIDATION OF THE PROPOSED METHOD

4.1. Geometrical description and FE modelling

The proposed method has been applied to a case study and analysed by the commercial FE code Abaqus to validate the hypotheses made on the loads system acting on the structure. The case study is a masonry bridge characterized by an arch with angle of embrace less than 180°, thickness of 0.8m and abutments 1.4m thick. The span of the arch is 10m and the rise 4m. The depth of the infill over the crown is of 1.0m (Fig. 4) and its friction angle is 30°. In order to better represent the infill horizontal pressure, a large portion of the backfill around the bridge has been considered in the numerical model. The dead and seismic loads have been applied in two different steps to investigate the corresponding effects of the infill horizontal pressure on the structure.

Regarding the boundary conditions, at the base of the abutments and of the soil both the displacement directions have been prevented, while in the points of the lateral right side the vertical displacements have been allowed. In the left side a rigid bound has been included with an unilateral contact between the two surfaces, in order to allow the separation of them in presence of the seismic actions. Moreover, particular attention has been devoted to the contact condition between the backfill and the structural arch. Also in this case, the application of a frictionless unilateral contact law has allowed describing the pressure effects on the abutments and the arch and, at the same time, to avoid local nonlinearities due to even small relative sliding between the surfaces.

Non-linear analysis has been performed in plane strain condition. An isotropic behaviour has been considered for the materials in the elastic range, while the classical Mohr-Coulomb (MC) criterion and the Concrete Damaged Plasticity (CDP) model have been used for the description of the fill and the masonry respectively beyond the elastic limit. In the CDP model, tension stiffening with softening behaviour has been considered by taking from the literature the points in the post-peak phase. In Tab. 3 the mechanical and geotechnical parameters are summarized.

4.2. Comparison between the proposed method and the FE results

Force-controlled numerical analyses have been performed by increasing the horizontal actions proportionally to the mass density up to the achievement of the convergence limit. In Fig. 5 the horizontal stress distributions developed inside the backfill has been shown, in which the coloured map corresponds to the ultimate step of the analysis. At the sides of the picture the graphs of the horizontal pressures at a distance of 1.4 m from the abutments have been reported both for the active (red line) and the seismic condition (blue line). Even if the presence of the thickness discontinuity at the conjunction between the arch and the abutments causes a stress concentration in the backfill, the value of the pressures at the base obtained by the numerical model (about 50 kN/m^2) are in a good agreement with the simplified proposed method, which consider a linear distribution reaching a maximum value at the base equal to 58.7 KN/m^2.

Regarding the pressures induced by the seismic actions, the left diagram highlights a good agreement with the hypotheses made in the proposed method.

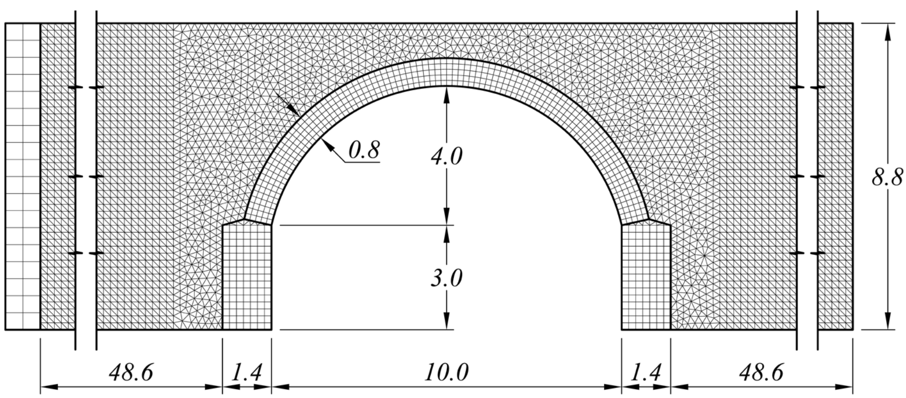


Fig. 4. *Geometrical dimension of the FE model (dimensions in m).*

Table 1. Mechanical and geotechnical parameters assumed in the analysis.

Material	Property	u.m.	value
Masonry	Unit weight	kN/m^3	20
	Elastic modulus	kN/m^2	1.5e+07
	Poisson's ratio	-	0.2
	Compression strength	kN/m^2	4500
	Tensile strength	kN/m^2	150
Backfill	Unit weight	kN/m^3	20
	Elastic modulus	kN/m^2	3e+05
	Poisson's ratio	-	0.2
	Friction angle	deg	30
	Dilation angle	deg	20
	Cohesion	kN/m^2	0.001

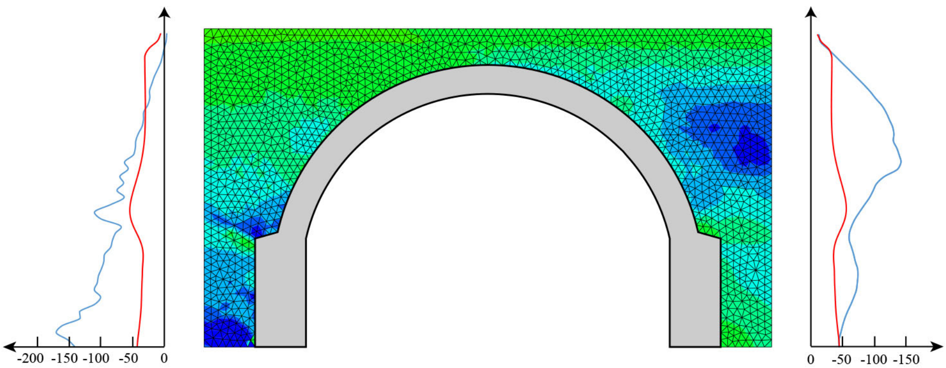


Fig. 5. Horizontal backfill pressures: active (red) and seismic (blue) conditions (values in KN/m^2).

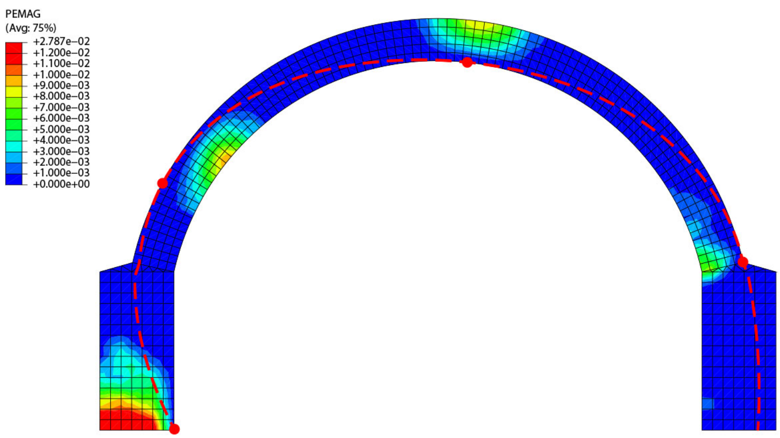


Fig. 6. Plastic strains and line of thrust with the limit analysis method (dashed red line).

The passive pressures seem, instead, having a quite different shape from the assumed one and it seems being function of the collapse mechanism. Nevertheless, the obtained overall structural response is consistent to the FEM results both in terms of the kinematics mechanisms (Fig. 6) and of the collapse multiplier. The limit analysis gives a value of μ=0.34, and the evolution of the plastic strain in the FEM model are included in a range of μ=0.326÷0.441.

5. PARAMETRIC INVESTIGATIONS

In order to evaluate the influence of the backfill on the bearing capacity of the bridge, a parametric investigation has been carried out by varying the backfill friction angle ϕ' and the abutment height h. The seismic load multiplier μ has been determined, through the numerical procedure described at paragraph 3, for each couple ϕ',h. The results, reported in Tab. 2, show an increasing seismic load multiplier for increasing values of the friction angle. In fact, the more the backfill is frictional, the more will be its positive contribution to the safety of the arch in its plane. Moreover, it can be noticed that μ decreases when the abutments become higher.

6. CONSOLIDATION TECHNIQUES

The relevant age of most of the existing masonry bridges, the increased traffic load acting on them, the deterioration progress and the higher safety level expected by the community, require repair or strengthening intervention on these structures. Several methods are nowadays available to repair or strengthen an existing bridge. The choice of the type of intervention depends on its efficiency against the faults that affect the bridge. In the following the most common repair and strengthening methods are examined with special attention to the seismic vulnerability of the masonry bridge in the longitudinal direction. The measures considered in this paper are the thickening of the arch by a concrete saddle at the extrados and the application of fiber-reinforced materials.

6.1. Arch thickening

Saddling can be considered the most common intervention to increase the load bearing capacity of a masonry arch bridge. It consists in the casting of a RC arch on the top of the existing arch. Suitable connectors allow the composite action between the existing arch and the new one. Conversely this method involves the complete removal of the fill with the consequent negative economic aspects.

The presence of the concrete saddle is modelled, in the limit analysis numerical procedure, by changing the thickness of the arch and consequently by modifying the resistance criterion. Let us denote by d_s the thickness of the concrete saddle. The analysis of the bridge has been carried out by considering an arch with a thickness equal to $s+d_s$, being $d_s = 0$ in correspondence of the abutments (Fig. 7). A parametric investigation has been carried out, starting from the case study previously analysed, by varying the thickness of the concrete saddle in the range [0, 0.4] m. The results are reported in Fig. 10. The red dot corresponds to the condition of absence of interventions and the dotted blue line represents the trend of the seismic load multiplier μ after the arch thickening.

It can be observed a linear increase of the seismic load multiplier until the value d_s = 0,15 m. By further increasing the thickness d_s a variation of the type of collapse mechanism results, namely there is a shifting of the hinge U from the right springing of the arch to the abutment. As a consequence, the resistance of the structure increases, but the intervention becomes less effective since it involves the arch only. In fact, a reduction of the slope of the blue line can be observed for values $d_s > 0{,}20$ m.

Table 2. *Seismic load multipliers from the parametric investigation.*

Abutment height h [m]	Backfill internal friction ϕ' [°] 25	26	27	28	29	30	31	32	33
2.50	0.31	0.32	0.33	0.34	0.35	0.36	0.37	0.38	0.40
2.75	0.30	0.31	0.32	0.33	0.34	0.35	0.36	0.37	0.38
3.00	0.29	0.30	0.31	0.32	0.33	**0.34**	0.35	0.36	0.37
3.25	0.28	0.29	0.30	0.31	0.32	0.33	0.34	0.35	0.36
3.50	0.26	0.27	0.29	0.30	0.31	0.32	0.33	0.34	0.35
3.75	0.25	0.26	0.27	0.28	0.30	0.31	0.32	0.33	0.34
4.00	0.24	0.25	0.26	0.27	0.28	0.29	0.30	0.32	0.33

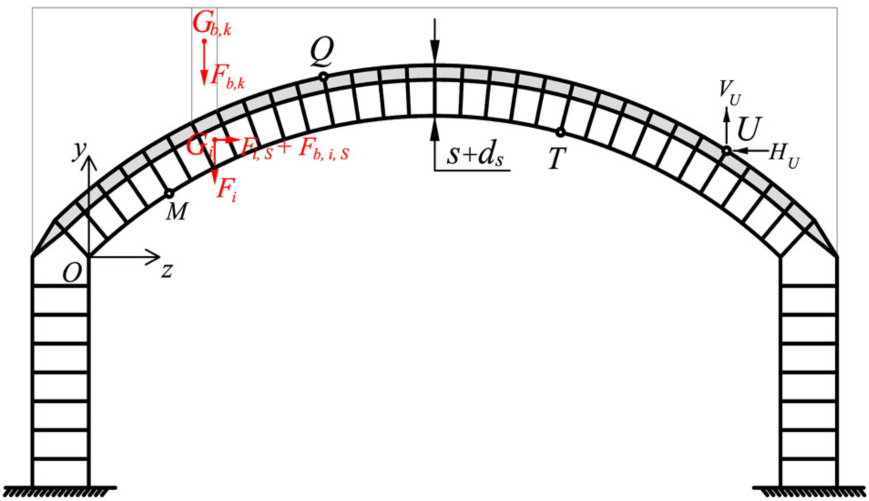


Fig. 7. *Consolidation by a concrete saddle at the extrados of the arch.*

6.2. Strengthening with FRP materials

Arch's reinforcement can be realized with full or partial length FRP strips placed separately or jointly at the extrados and at the intrados of the arch barrel. In historical structures this intervention should be generally limited to the extrados since it is invisible once completed. Nevertheless, even in this case, this consolidation technique requires the complete removal of the fill. The application of fiber-reinforced materials at the extrados permits the line of thrust to fall outside the arch at the intrados [17]; this can be considered as an additional tensile strength of the masonry, which is proportional to the resistance F_f of the fiber and to the normal force acting at each joint (Fig. 8 and 9).

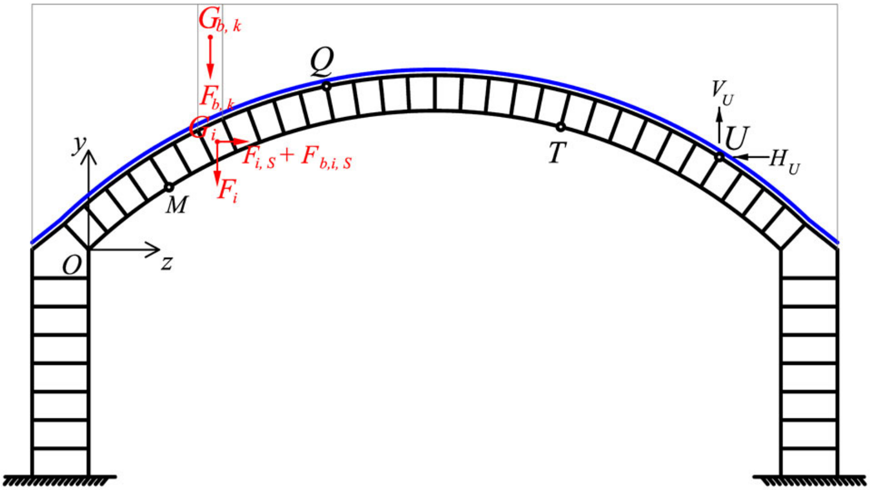
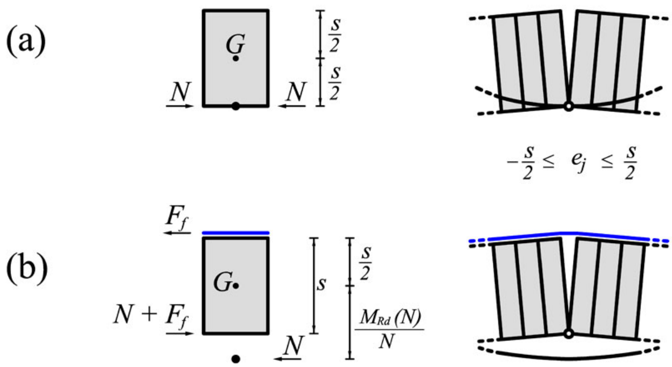


***Fig. 8.** Consolidation by application of fiber-reinforced material at the extrados of the arch.*



$$\frac{M_{Rd}(N)}{N} = \frac{Ns/2 + F_f s}{N} \qquad -\frac{M_{Rd}(N)}{N} \leq e_j \leq \frac{s}{2}$$

***Fig. 9.** Resistance criterion in the case of (a) unreinforced arch and (b) strengthening with fiber-reinforced materials adapted from [17].*

A parametric investigation has been conducted on the case of study by varying the resistance F_f of the fiber. The results are represented in Fig. 10. The red dot corresponds to the initial condition of the structure without interventions and the dashed black line shows the trend of the seismic load multiplier μ depending on F_f. Until the value F_f = 210 KN/m a linear increasing of μ can be observed. By further increasing the tensile resistance of the fiber, a variation of the type of collapse mechanism can be observed: the shifting of the hinge U from the right springing of the arch to the abutment produces an increment of the seismic load multiplier and a variation of the black line slope.

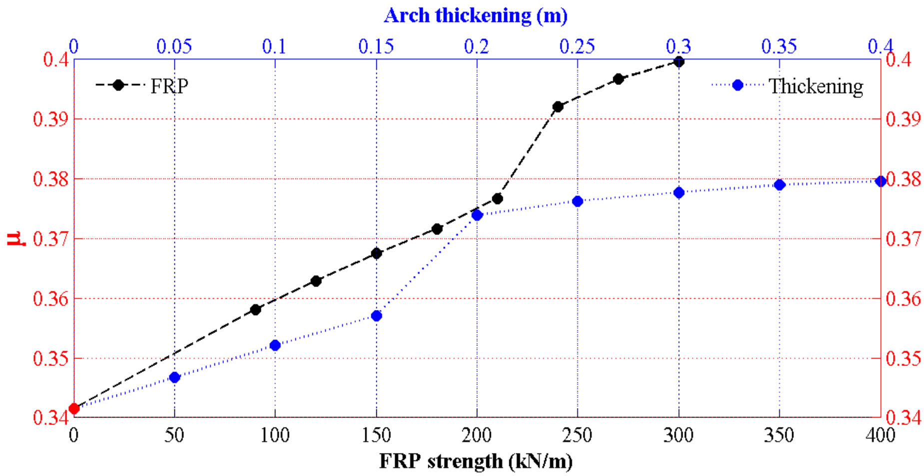


Fig. 10. *Seismic load multiplier vs thickness of the concrete saddle and resistance of the fiber.*

7. CONCLUSIONS

In this paper a numerical procedure for the seismic assessment of masonry arch bridges has been developed referring to limit analysis.

In a first phase, the seismic load multiplier leading the structure to the collapse has been evaluated. Particular attention has been dedicated to the simulation of the inertial effect of the backfill and to the definition of the lateral soil pressures. Active and passive pressures, evaluated following the Rankine theory, have been considered to act on the arch and abutments. The seismic increment of active pressure has been taken into account by referring to the Mononobe-Okabe approach. A Finite Element Model of a case of study has been developed in order to validate the numerical procedure and the soil pressures trend. The results are in good agreement, confirming the numerical procedure's validity. In particular, it was found that the more the backfill is frictional, the greater will be the bearing capacity of the bridge in its plane. Morover, slender abutments make the structure more vulnerable to the seismic actions.

In a second phase, the effect of two interventions on the collapse behaviour of the arch bridge has been analysed. The arch thickening by means of an extrados concrete saddle and the application of fiber-reinforced materials have been considered. An increasing seismic load multiplier has been obtained by increasing both the thickness of the concrete saddle and the tensile strength of the fiber material. A reduction of the effectiveness was found when the interventions on the arch modify the collapse mechanism, by shifting the extreme right hinge from the arch to the abutment.

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