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A RECONSTRUCTION HYPOTHESIS OF COLLAPSED ARCHAEOLOGICAL MASONRY BARREL VAULTS EMPLOYING FRICTIONAL BEARINGS, WITH APPLICATION TO THE CASE STUDY OF THE *GALLERIA DELLE VOLTE CROLLATE*, IN ROME

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Abstract. *This paper presents a study of a reconstruction hypothesis of collapsed masonry vaults with archaeological value, by taking advantage of the features of frictional bearings, either flat or curvilinear. Such study finds a natural application to the case of the "Galleria delle Volte Crollate", which is a collapsed rubble masonry barrel vault, dating back to the I century BC and placed in the Roman Forum, in Italy. The proposed intervention strategy is based on the need to fulfill both philological and structural criteria. As to the former, the solution has to reconstruct the original geometry, while suggesting the historical memory of the occurred collapse. As to the latter, both geometrical and tribological features of the isolation system have to be designed in order to fulfill both static and seismic ultimate limit states. This design can be carried out by means of a parametric study. In fact, based on the case-study herein dealt with, parametric studies are conducted in order to single out the values of the parameters corresponding to the optimal solution. Non-linear dynamical analyses with site-spectrum compatible accelerograms are performed by means of the commercial Finite Element software SAP2000®. The innovative aspects of the proposed solutions are highlighted along with limits and possible further developments.*

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1 INTRODUCTION

The Italian Archaeological Heritage presents numerous buildings roofed by masonry vaults. Many of these are still standing, even though in need for a retrofitting intervention due to intrinsic structural deficiencies, while many others collapsed, either partially or totally. In this scenario, it is necessary to distinguish the type of intervention for either case, adopting the most suitable design strategy, given the boundary conditions.

The current Italian Building Code [1], as to the design of retrofitting interventions on existing buildings, does not provide information on the choice of intervention type on vaults. An applicative document complementing the Italian Building Code [2] deals with the topic more extensively, suggesting methods both to consolidate the vaulted roofs and to reduce the stress level in its masonry components. In particular, this document gives some indications about the interventions. It discourages the construction of a reinforced concrete vault at the extrados of the existing one, since it increases masses and introduces a clear discontinuity in terms of stiffness in the transversal cross section, thus altering the original mechanical behavior. The heavy filler should be replaced by either stiffening masonry ribs (*frenelli* in Italian) or lightweight material such as expanded clay, for example. Typical overturning of the piers can be avoided by means of either steel tie rods or buttresses. These latter are also useful to maintain the pressures within the central core of inertia. The use of Fiber Reinforced Polymer (FRP) strips is also allowed. In more general terms, the restoration theories are promoters of interventions based on the following *applicative criteria*: a) minimal intervention, b) reversibility, c) recognizability and d) compatibility [3,4]. These criteria substantially imply that the structural strengthening intervention should not modify the original mechanical behavior of the monument and the added structural elements should be removable without bringing any damage. Furthermore, the assessment of historical buildings should be carried out by means of methods developed specifically for the given monuments, in order to reduce the risk to propose invasive interventions, which might alter their structural behavior and permanently modify their cultural value.

The most recent seismic events in Italy (Abruzzo 2009; Emilia Romagna 2012; Lazio-Marche 2016) confirmed the limits of some interventions that, based on the alleged need to safeguard the traditional construction techniques, obstinately aim at rebuilding structures that were clearly affected by structural deficiencies *ab origine*.

Among the innovative techniques, the implementation of base isolation systems for the reduction of the seismic action on buildings has become a common alternative to conventional strengthening measures [5]. It has been estimated that altogether a total of approximately 16,000 structures have been protected in different parts of the world by seismic isolation, energy dissipation and other anti-seismic systems [6]. Most of them are located in Japan, although they are more or less numerous in over 30 other countries. A base isolation system consists of an isolation layer placed between the building and the ground, which, in case of dynamic actions, decouples the building and the soil motions. Smaller inertia forces, smaller inter-story drifts, and smaller seismic forces in structural elements therefore characterize base-isolated buildings. Techniques have been developed that can be used to insert base isolation under existing buildings with some additional costs, which might be justified especially in the case of historic buildings with extremely high or even inestimable value (*e.g.* [7-12]). However, these retrofitting interventions, most of which realized abroad, end up being very invasive due to the necessity to create a rigid plane both at the intrados and at the extrados of the so-called isolation plane. In fact, in order to fulfill such need, in most of those cases, a thick reinforced concrete slab was realized at the extrados of the isolation layer. Such heavy interventions are not allowed by the Italian building Code that only prescribes seismic amelioration for historical buildings.

In some cases of collapse, when the original vault element was affected by intrinsic and inherent structural problems, reconstructing faithfully to the original features may not make sense. In fact, in the extreme cases, such intervention philosophy may imply: a) rebuilding the deficient vault, and b) retrofitting it, according to the criteria and with the techniques above, before considering it safe to be open to the visitors. In such cases, a reasonable reconstruction intervention may be based on the following two philological criteria: 1) recreate the original geometric shape, and 2) keep memory of the history of the monument, including the collapse due to the wrong original conception. Moreover, such philological approach may be implemented exploring the possibility to use both new materials and innovative techniques, which would help ameliorate the structural behavior of the monument, ensuring a longer life and thus preserving it for future generations.

In particular, in the present work, with reference to the case of lowered barrel vaults, we have explored the possibility of building a self-supporting roof to rest on the pre-existing wall structures and to which only the cortical portions of the heavy blocks resulting from the collapse will be fastened. A steel truss may realize the structure of the roof. It must be stressed that the adoption of steel structures in the retrofitting interventions of ancient masonry structures was already accepted and implemented by the community of restoration practitioners and academics [13]. The bearing of such structure may be of the frictional type. The transversal cross-section may be either flat or curvilinear, while the longitudinal dimension may be either point-wise or continuous. The transverse cross-section is dictated by the zone seismicity: if seismicity is null or negligible, the flat bearing can be used otherwise, if the seismicity is high, the spherical bearing can be used. This latter eventuality contemplates the use of the so-called friction pendulum devices that should ensure re-centering after the motion. The longitudinal dimension of the bearing is dictated by the static combination of loads and it is more or less extended (*i.e.*, point-wise or continuous) as function of the need to limit the relevant value of the vertical stress transmitted to the underlying masonry. The value of the friction coefficient, and in turn the corresponding pair of materials defining the sliding interface, is dictated by the necessity to limit, taking advantage of the rigid-perfectly plastic Coulomb-type frictional law, the maximum value of the horizontal force transmissible to the underlying wall in correspondence of the seismic load combination.

The present work is divided in three main parts. The first part shows some examples of reconstruction and the proposed strategy of intervention by means of frictional bearings. The second part describes the case study of the *Galleria delle Volte Crollate*, placed in the central archaeological area of Rome, in Italy. The third part describes the mechanical model of the structure and the parametric studies, carried out by means of a non-linear dynamic analysis, to optimize the design. Such analyses were carried out by means of Finite Element model developed in the commercial software SAP2000® [14]. The so-called *Galleria delle Volte Crollate* is an important archaeological monument located in the Roman Forum, atop the *Palatinum Hill*.

2 INTERVENTIONS IN CASE OF PARTIAL OR COMPLETE COLLAPSE

2.1 Examples of reconstruction

Ancient monuments, especially after the collapse of some of their parts, show structural deficiencies that represent singular situations and whose solutions require special design implications and a technological creativity in the intervention strategy. One example is the "Oratorio di San Filippo Neri" in Bologna (Italy), which was restored in 2000. In fact, in 1944 the Oratory was bombed, and the roof, the vaults, the dome were destroyed (Fig. 1a) [15]. The restoration design redefines the original shape of the dome and the vaults, building a wooden structure made of supporting ribs and slats (Fig. 1b). The wooden structure integrates the original one collapsed, properly reinforced by means of steel rods and carbon fibers. The qualities of the wood, as a material fully compatible with the restoration philosophy, are numerous: lightness, mechanical resistance, easy workability and reversibility (dry technology).

Figure 1: Examples of either partial or complete reconstruction: *San Filippo Ner*i Oratory in the city of Bologna a) soon after the 1944 bombing and b) after restoration of 2000; c,d) *San Peter's Basilica* in Siracusa; e,f) *Domus Tiberiana* in Rome; g,h) huge vault of *Villa Adriana* Stadium in Tivoli .

The paleo-Christian Basilica of "San Pietro" in Syracuse is a further example of a restoration design that uses the wood material to re-propose the architectural geometry of a structural element no longer in existence (Fig. 1c,d) [16]. The false ceiling in wooden slats is set up at the height of the paleo-Christian age barrel vault without ever touching the perimeter wall (Fig. 1d) and is supported by the overhanging truss roof. This clearly follows the criteria of minimal intervention and is easily reversible.

For the “Domus Tiberiana” in Rome (Fig. 1e,f), the reconstruction of some masonry vaults was adopted as a strategy for structural amelioration of the monument. The monument had been affected by the collapse of its vaults, which have been completely reconstructed during the restoration intervention due to the favorable boundary conditions. Such reconstruction was carried by firstly assembling a wooden-rib supporting structure (Fig. 1e) [17].

In a typically archaeological context, a restoration example, similar in terms of structural problems to the case study presented in the next section, is the “Voltone” at the “Stadium of Villa Adriana” in Tivoli [13]. Before the restoration, some large barrel vault blocks of the monument lay on the ground due to the collapse of the piers originally supporting the roofing system (Fig. 1g). The restoration had foreseen: 1) the reconstruction of two new reinforced concrete pillars, 2) the construction of a wooden rib, 3) the repositioning of the blocks on the rib, and 4) the integration of the missing parts with new material similar to the ancient vaults (Fig. 1h). This solution was made possible thanks to the new pillars reconstruction that offer maximum guarantees of resistance to the compressive stresses imposed by the heavy blocks.

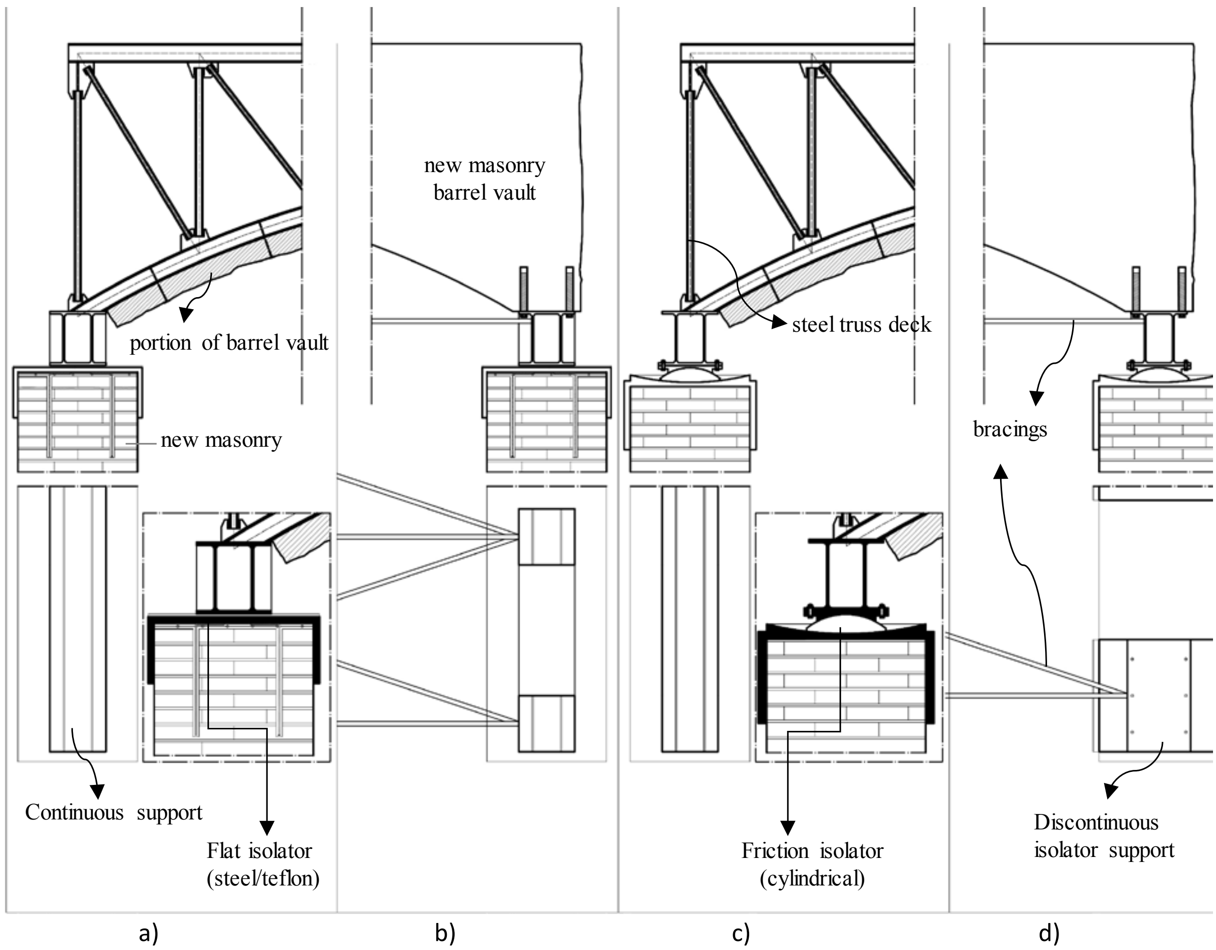


Figure 2: Frictional bearings’ possible design solutions for both steel truss deck (left) and new masonry vault (right): a) long, and b) point-wise flat bearing, c) long and d) point-wise cylindrical SFP.

2.2 *Proposed solutions based on frictional bearings*

In the case of collapsed vaults, when the supporting walls are sufficiently rigid not to necessitate the construction of an invasive rigid diaphragm under the isolation layer, the isolation technique may be adopted together with the reconstruction of the vaulted roof by new materials that may reduce the masses and provide a rigid plane.

In some particular cases, as in the case study herein presented, due to the already stiff understructure, the possibility of seismic isolation can be explored as a way to intervene also on ancient masonry structure, to fulfill a twofold need: 1) bringing the least disturbance to the ancient structures, and 2) decoupling them from the earthquake-induced ground motion.

Some innovative proposals, contemplating for the first time the use of frictional isolation bearings in an archaeological environment, were herein developed hypothesizing two possible intervention scenarios: 1) the reconstruction of a new masonry vault, and 2) spatial repositioning of vault portions by means of a steel-truss-deck supporting structure. Only some portions of the old masonry structure may be hung to the steel truss deck.

A first design solution (Fig. 2a,b) proposes the insertion of flat sliding devices, with Poly-Tetra-Fluor-Ethylene (PTFE) and stainless steel interface, in order to limit the horizontal force transmitted by the new structure to the underlying supporting masonry during an earthquake, by seizing the rigid-perfectly plastic Coulomb's friction behavior. Since such devices may not guarantee the necessary displacement re-centering after a seismic event, a second design solution (Fig. 2c,d) foresees the use of devices characterized by a circular shape. These bearings, known as Friction Pendulum Devices (FPDs), are available in the market, for the case of point-wise bearing, and are composed of spherical sliding surfaces (*e.g.*, [18,19]). They are supposed to warrant re-centering due to the tangential component of the gravity load. In the present work, we are assuming that they can be produced with a cylindrical sliding surface (Fig. 2). In fact, as already stated above, given the presence of long walls, these can support the seismic horizontal force directed along their length, while the orthogonal action should be limited as function of the masonry strength. Such limitation can be guaranteed by suitably selecting the friction coefficient and in turn, the materials constituting the sliding pair.

As to the masonry solution, regardless of the type of the bearing cross section, either flat or cylindrical, the necessity to recreate an horizontal rigid plane at the extrados of the isolation layer, would imply the use of a horizontal steel truss made by tie rods (Fig. 2b,d).

3 FROM DISINTEGRATION TO STRUCTURAL RECOMPOSITION: THE CASE STUDY OF THE "GALLERIA DELLE VOLTE CROLLATE"

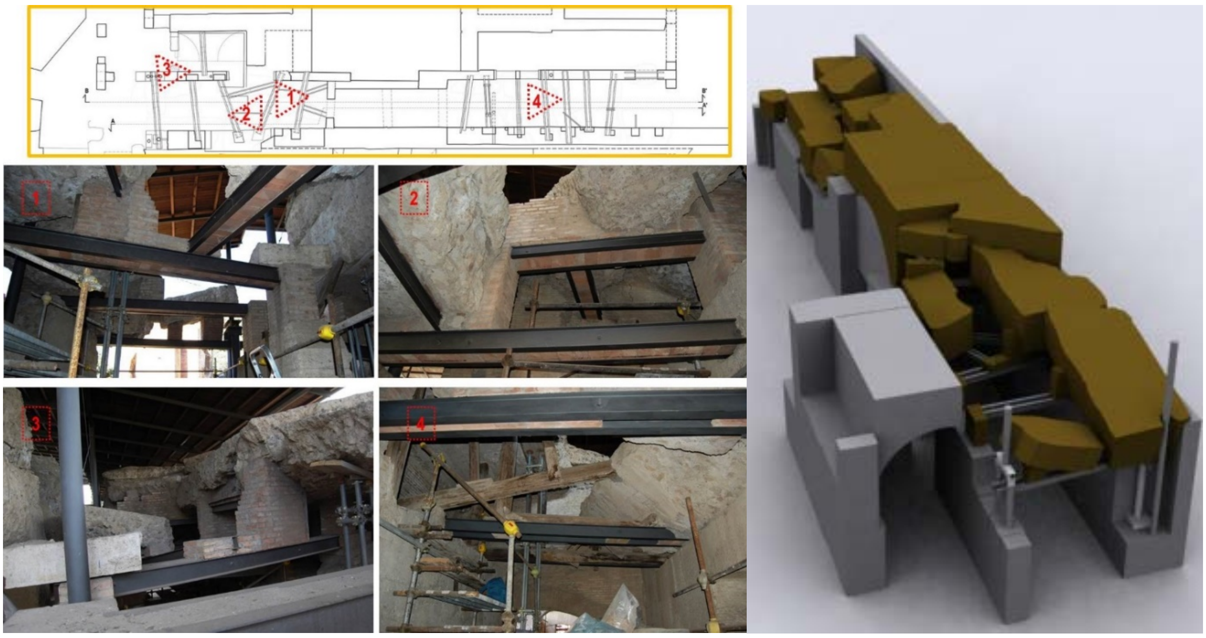
3.1 *Ante Operam*

The so-called "Galleria delle Volte Crollate" (The Collapsed Vaults Gallery) is located in the central archaeological area of Rome, the Roman Forum, precisely in the south-west area of Augustus' House, an important archaeological site atop the *Palatinum* hill (Fig. 3).

The Gallery was originally roofed by a depressed barrel vault made by a conglomerate composed of irregularly shaped tuff stones and pozzolanic mortar, according to a technique called *opus caementicium*. The vault collapsed, disintegrating in large conglomerate blocks, in an unspecified time of its history, probably due to an earthquake [20]. These resulting blocks were rearranged in the second half of the twentieth century (between 1960 and 1980) by a complex and heterogeneous structural system comprising steel beams, squat brick masonry walls, and wood struts (Figs. 4,5). Only some portions of the vault are still standing in their original positions. The structures beneath the level in question, including the foundations, have not been excavated yet.

Figure 3: Urban localization: general view of the central archaeological area of Rome.

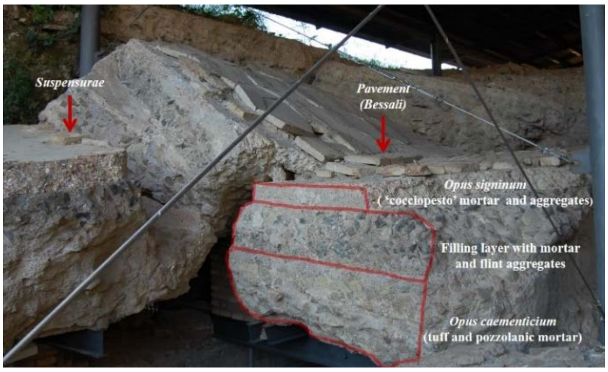
In plan, the Gallery is ideally divided into three parts (Fig. 6): a) the first one of length $12{,}41\ m$ and variable width between $3{,}27\ m$ and $3{,}44\ m$, b) the second one is $7{,}14\ m$ long and $2{,}60\ m$ wide, c) and the third one of length $15{,}81\ m$ and variable width between $3{,}29\ m$ and $3{,}36\ m$.



a) b)

Figure 4: Previous retrofit intervention (XX century): a) plan view with optical cones of images, details of the steel beams, squat brick walls and wooden struts, and b) rendering.

The depressed barrel vault constituting the roof is characterized by the following geometrical features: its spring line is $4{,}45\ m$ and its rise is $0{,}59\ m$. For what concerns the materials: the "structural part" of the vault is made of tuff conglomerate and pozzolanic mortar, while the filling is composed of two layers. The former is made of mortar and flint aggregates and the latter, a thinner layer, in *cocciopesto* aggregates (Fig. 5a). The extrados shows traces of the so-called *suspensurae*, that is, short brick walls supporting the pavement and spaced apart in order to create a ventilated interspace, and a pavement in *bessali*, a typical Roman brick, that leaves guess that the gallery was also accessible at the extrados (Fig. 5a).



a)

b)

Figure 5: Previous retrofit intervention (XX century): a) plan view with optical cones of images, details of the steel beams, squat brick walls and wooden struts, and b) rendering.

3.2 *Structural restoration design*

After having realized the inefficacy of the previous retrofitting intervention, described in the previous paragraph, different alternative retrofitting strategies were contemplated [20] at a preliminary stage. The selected retrofitting strategy, among these alternatives, consists of a roof made by a three-dimensional steel truss deck, with flat walkable extrados and barrel vault-shaped intrados, which is meant to recreate the same cylindrical surface of the original barrel vault. Such steel structure lies on the two existing longitudinal walls by means of flat and continuous isolation bearings, whose interface is composed of PTFE and stainless steel. Only the cortical part of the resulting masonry blocks will be preserved while the remaining parts will be discarded. The cortical layers, at both the intrados and the extrados, will be cut into easily maneuverable pieces that will be fastened to the intrados of the steel structure. This solution was chosen since it fulfills both conservation needs and structural safety. As to the latter, it guarantees: 1) minimum disturbance to the existing ancient masonry walls due to the low bearing pressure, and 2) safety against a transversal earthquake, thus eliminating one of the defects that may probably have caused the collapse of the original vault. Since the new roof of the gallery is seismically isolated, it can be designed for gravity load combinations only. This is in perfect analogy with what happens in the seismic design of isolated bridges, in which the overall geometry and the pier/deck sections are designed for non-seismic load conditions only [21]. Thus, the only design variable is represented by the Isolation System (IS). Since we know the maximum transversal force that the longitudinal supporting walls can stand, we can design the isolation system accordingly. The details of the resulting structure are described in this paragraph, while further information about the calculations can be found elsewhere [21].

The retrofitting strategy of the Gallery involves the construction of a bridge-like steel truss deck whose cross-section is such as to recreate the same intrados profile of the original vault and to which the blocks of vault are fixed. The extrados of this new structure will be an accessible panoramic viewpoint.

This solution was designed to satisfy the following objectives: 1) repositioning the blocks of the vault; 2) optimizing all operations and the various processes to be performed in safety; 3) causing the minimum disturbance to the supporting ancient walls, distributing the stresses exerted by the steel structure, and 4) achieving a reversible intervention.

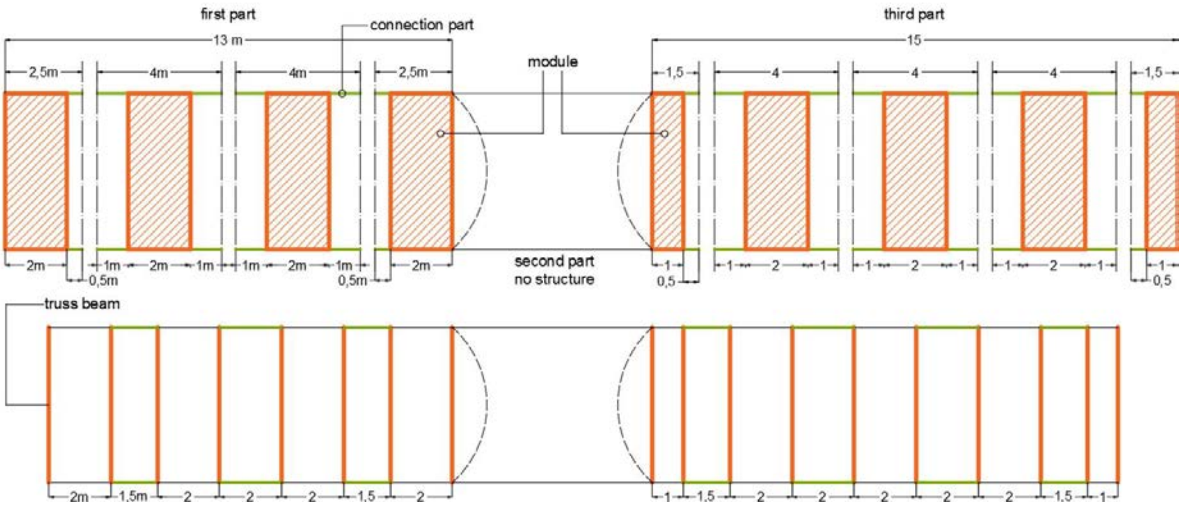


Figure 6: Modular steel truss deck adopted for the restoration design of the "Galleria delle volte crollate": plan view of the several prefabricated modules assemblage *in situ*.

In order to both facilitate and speed up the construction process, thus reducing the intervention costs, the gallery may be realized by a partially prefabricated steel truss deck (Fig. 6). This latter may be obtained by the *in situ* assemblage of prefabricated modules, each composed of two transversal steel truss beams. In the basic module, the two beams are spaced apart 2 m, while in correspondence of the gallery extremities, such distance is less than 2 m (Fig. 6).

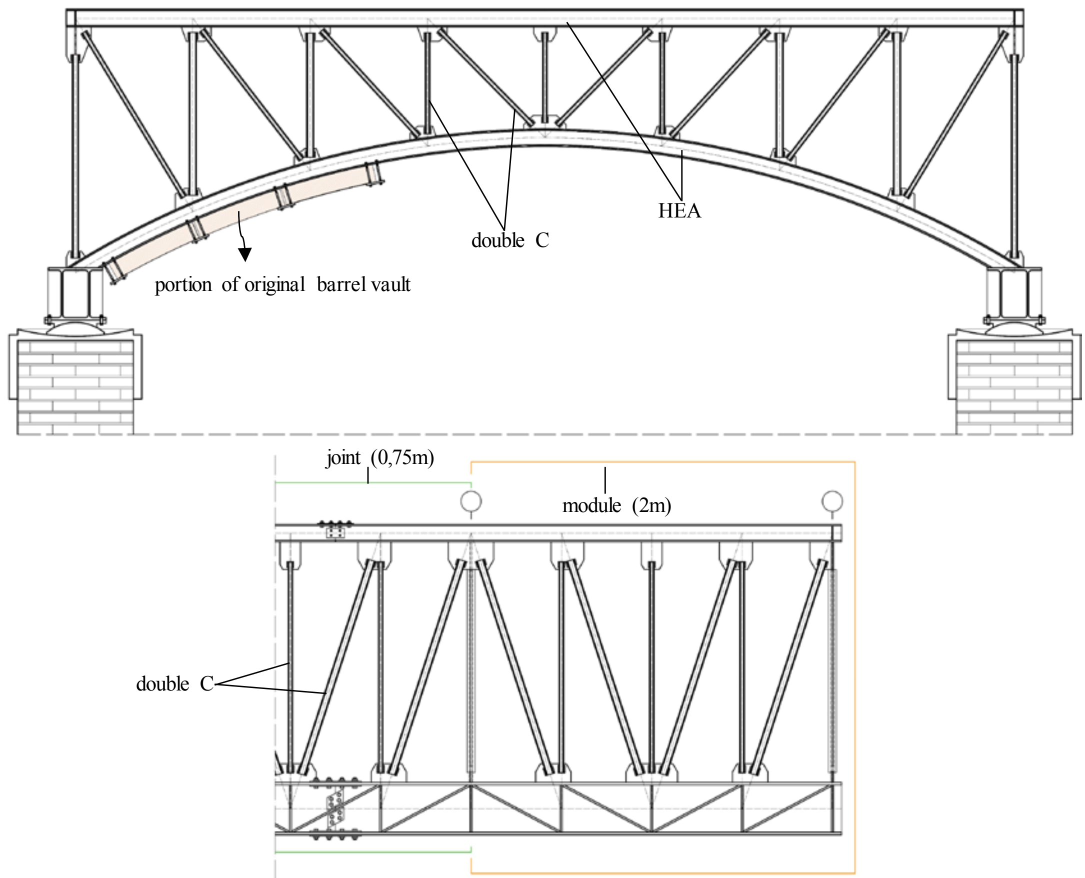
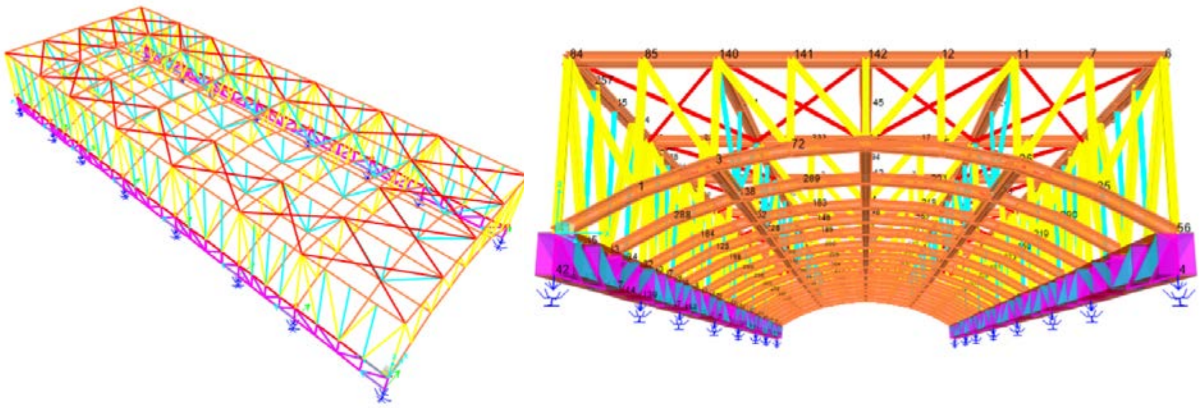


Figure 7: Restoration design of the "Galleria delle volte crollate":
transversal cross section of the steel truss deck and detail of the longitudinal joint between adjacent modules.

The principal truss beams have diagonals and rods made with double "C" profiles and connected to the lower and the upper current HEA type (Fig. 7). The longitudinal beams are made by much more rigid diagonal rods than the transverse struts. The deformation of these latter allows to load the diagonals, so distributing the stresses uniformly to the underlying masonry (Fig. 7). Furthermore, the longitudinal supporting beam is a composite box girder (similar to the runways of bridge cranes), which is very rigid so as to further contribute to the stress distribution (Fig. 7).

The cutting of the blocks of vault in cortical parts, both at the intrados (curvilinear) and at the extrados (flat) fixed respectively to the upper and lower steel structure, allows to considerably reduce the loads (Fig.7). The cortical layers portions sizes are dictated by both maneuverability and self-bearing capacity. The whole structure will simply lie on the ancient masonry, by means of the PTFE/stainless steel interface. In this way, the new roof may be easily removed, thus adequately fulfilling the reversibility criterion.

The design envisages the following phases: 1) moving the blocks outside the gallery perimeter, 2) positioning the prefabricated steel truss modules on the longitudinal supporting walls and assembling them on site, 3) cutting of the cortical part of the blocks, both intrados (curvilinear) and extrados (flat), 4) positioning and fastening of the cortical parts, whose dimensions may be such as to allow bare hands movement, 5) construction of the walkable floor at the extrados. This latter will be realized by a reinforced concrete slab at the extrados, cast as completion of a cold-formed profiled sheet.



a) b)

Figure 8: Finite Element Model (FEM) of the new roof's steel truss deck: a) numerical model and b) extrusion details.

4 MECHANICAL MODELLING

The structural model of the new reticular roof was developed with SAP2000® to both optimize the calculation procedures and perform design and static verifications of all steel elements (Fig. 8).

Sliding isolation devices reduce the seismic force transmitted to the extrados of the lateral longitudinal walls by limiting the shear transfer across the sliders. The attention is focused on the transversal response of the steel truss, which is the most critical one, due the lower strength offered by the walls cross-section. The total mass of the new superstructure amounts to $m_{tot} = 60{,}21\ kNsec^2/m$, whereas the original masses (masonry vault) amounted to $m_{tot} = 199\ kNsec^2/m$, which is less than one-third.

Masses in the seismic combination [1] were evaluated as follows:

$$G_{k1} + G_{k2} + \sum \psi_{2j} \cdot Q_{kj} \tag{1}$$

where: G_{k1} and G_{k2} are the characteristic values of the structural and non-structural dead load, respectively; Q_{kj} is the characteristic value of the j-th variable load and ψ_{2j} is the coefficient to obtain the quasi-permanent load value. As to the variable loads, we assumed the presence of people only, thus implicitly assuming as highly unlikely the concomitance of people and snow. Moreover, we assumed a characteristic value of $2.0\ kN/m^2$ since such structure will be subject to the weight of a limited number of people, thus excluding the possibility to use it for crowded events.

4.1 *Assumptions*

As first approximation, the following assumptions were made:

1) for the time being, due to the squat form of the longitudinal ancient masonry walls, their movement was considered uncoupled from the deck's;
2) attention was focused on the modular structure, thus neglecting any collaboration between adjacent unit-length wall panels;
3) since the masonry walls have a much higher strength and stiffness in the longitudinal direction, only the transverse earthquake was considered;
4) due to the transversal vicinity of the two longitudinal walls, the seismic action at their bases is considered as synchronous, thus allowing the assumption of the in-phase oscillations of the walls.

As to the third item: the cylindrical FPD will be outfitted with a longitudinal retainer (Fig. 2) that will substantially allow the transfer of the longitudinal earthquake-induced force to the underlying masonry. The vertical contact surface between the FPD and the retainer will be provided with a very low friction coefficient ($\mu_d \cong 0{,}5\%$) by means of a lubricated liner, thus limiting the interaction between pad and retainer.

4.2 *Typical values of friction coefficient*

In order to single out the most suitable value of the friction coefficient and, in turn, of the corresponding materials constituting the relevant kinematic pair, the typical values for typical materials were retrieved from the literature (Table 1) [23-26].

Friction coefficient	**Low velocity** μ_s	**High velocity** μ_d
Metal on metal	0,15-0,60	0,60
Stainless steel – PTFE	0,01	0,08-0,18
Stainless steel – Graphite	0,10	0,10
Stainless steel – GF-PTFE	0,01	0,01
Stainless steel – UHMWPE	0,02-0,05	0,025-0,055

Table 1: Typical ranges of values of the friction coefficient for kinematic pairs.

The range of values for the friction coefficient was distinguished between low velocity and high velocity, contemplating values that can be met with materials used in practice so far. For instance, μ_d can vary from 0,01, in the case of lubricated PTFE-stainless steel interface, by using the Glass-Filled Teflon (GF-PTFE) [24,25], up to 0,6 in case of steel-steel. Among the intermediate values, 0,1 can be obtained by using graphite-stainless steel interface. Even though in most applications the sliding kinematic pair is composed of stainless steel and PTFE, other materials are being applied, such as for instance Ultra High Molecular Weight Poly-Ethylene UHMWPE [27]. This latter is reported to provide outstanding properties as to: a) load bearing, b) resistance to wear, c) stability, d) durability, e) absence of the stick-slip phenomenon, f) low ratio between static and dynamic friction [27]. UHMWPE material, similarly to what happens for the more conventional PTFE, can be employed in the lubricated version, by means of superficial recesses containing the lubricant.

The value of the dynamic friction coefficient depends on both velocity and bearing pressure. However, for the range of velocity values expected in seismic applications, the velocity dependence of μ_d is negligible. On the contrary, the loading pressure dependence is not negligible and in particular μ_d decreases as the vertical load increases. For the time being, and for the sake of brevity, we neglect such dependence.

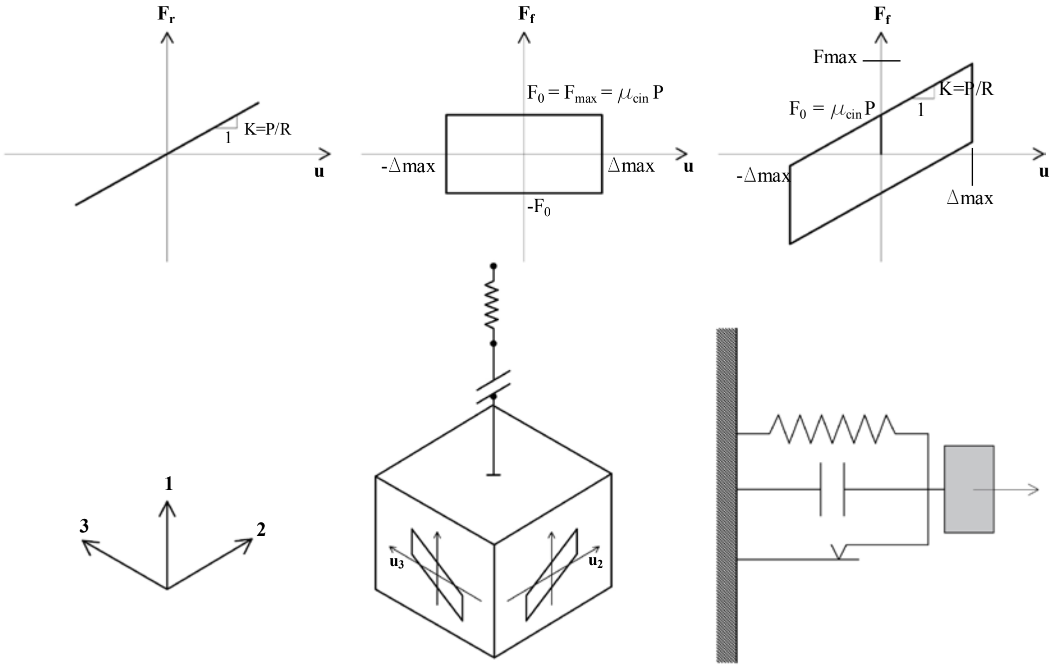


Figure 9: Modeling of the bearing through a Non-Linear Link.

4.3 *Adopted rheological model of FPD*

The bearing was modelled by a friction pendulum isolator Non-Linear Link (NL-L) available in SAP2000® (Fig. 9). This is a biaxial friction-pendulum isolator with the following features: a) coupled friction properties for the two shear deformations, b) post-slip stiffness in the shear directions due to the pendulum radii of the slipping surfaces, c) gap behavior in the axial direction, and d) linear effective-stiffness properties for the three moment deformations.

This element can also be used to model gap and friction behavior between contacting surfaces by setting the radii to zero, indicating a flat surface. In fact, for the longitudinal direction of the deck, due to the adoption of a cylindrical SFD, we assumed $R_3 = 0$.

The depth of the pad (Fig. 7) was assumed equal to $0{,}3\ m$. The length of the pad, is equal to the cylindrical FPD length so that it changes accordingly. In order to evaluate the mechanical properties and tributary masses for the relevant NL-L, the steel prism equivalent to the pad and the upper sliding plate was considered. For this latter, the mechanical properties to be input in the software were evaluated as indicated hereinafter.

Elastic shear stiffness k_2, for the direction transversal to the gallery (u_2), and axial stiffness, were evaluated based on the elastic deformability of the equivalent parallelepiped (Fig. 11) as:

$$k_2 = G_s \cdot \frac{d_i \cdot L_i}{h_i} \tag{2}$$

$$k_1 = E_s \cdot \frac{d_i \cdot L_i}{h_i} \tag{3}$$

where: E_s and G_s are the axial and shear elasticity modulus, respectively; h_i, d_i and L_i are the height, depth (along u_2) and length (u_3) of the FPD equivalent parallelogram.

Thus, the vertical gap element (Fig. 9) has the following equation:

$$F_{u1} = P = \begin{cases} k_1 \cdot u_1 & if \quad u_1 < 0 \\ 0 & otherwise \end{cases} \tag{4}$$

The mass of the FPD was evaluated as follows:

$$m_{SFP} = m_{NL-L} = \gamma_s \cdot V_{SFP} \tag{5}$$

where: γ_s is the steel unit weight, and V_{SFP} is the volume of the equivalent parallelepiped.

The rotational inertia around the i-th local axis:

$$J_i = \frac{1}{12} \cdot m_{SFP} \cdot (a_i^2 + b_i^2) \tag{6}$$

where: a_i and b_i are the dimensions of the cross section orthogonal to the i-th axis.

Equivalent linear stiffness:

$$k_e = N_{Sd} \cdot \left(\frac{1}{R} + \frac{\mu_d}{u_{max}}\right) \tag{7}$$

and equivalent viscous damping ratio:

$$\xi_e = \frac{2}{\pi} \cdot \frac{1}{\frac{u_{max}}{\mu_d \cdot R}} \tag{8}$$

where: u_{max} is the maximum value of the displacement induced by the earthquake and R is the radius of the sliding concave surface. As to u_{max}, though an iterative procedure should be carried out in order to obtain the right value, the average value ($u_{max} = 8{,}0\ cm$), equal to half of the maxim capacity $u_{Rd} = 16\ cm$ was herein assumed.

The lateral force provided by a FPD is given by (Fig. 9):

$$F = \frac{N_{Sd}}{R} \cdot u + \mu_d \cdot N_{Sd} = k_{ps} \cdot u + \mu_d \cdot N_{Sd} \tag{9}$$

where: N_{Sd} is the weight of the structure, R is the FPD curvature radius, u is the lateral displacement, μ_d is the dynamic friction coefficient, which also depends on the velocity and applied pressure and k_{ps} is the post-sliding stiffness.

The dependence of the friction coefficient on the value of the velocity was modelled as follows:

$$\mu \quad = \mu_d - (\mu_d - \mu_s) \cdot \exp(-r \cdot \dot{u}) \tag{10}$$

where: r is a rate parameter, herein assumed equal to 0,8, that defines the smoothness of the exponential curve describing the dependence of the friction coefficient on the velocity.

4.4 *Local-spectrum-compatible natural accelerograms*

Rigorously, the non-linear dynamic analyses should be carried out adopting seven natural accelerograms whose mean spectrum matches closely the site spectrum. The generation of the seven accelerograms was carried out by means of the REXEL software [28]. The elastic site spectrum was evaluated adopting the following values for the input parameters [29]: a) class soil A; b) topographic category $T1$; c) nominal life 50 years, and d) functional class II. As to this latter, we have assumed that, given the archaeological nature of the monument, it is reasonable to foresee a normal crowd. The site spectrum was evaluated for the Life Safety Limit State (LSLS) as indicated by the MiBACT document [30]. For the sake of brevity, only the numerical results concerning one of the seven selected accelerograms, that is the Umbria-Marche 1997 earthquake, are presented. The LSLS site spectrum and the relevant natural accelerogram are depicted in Fig. 10.

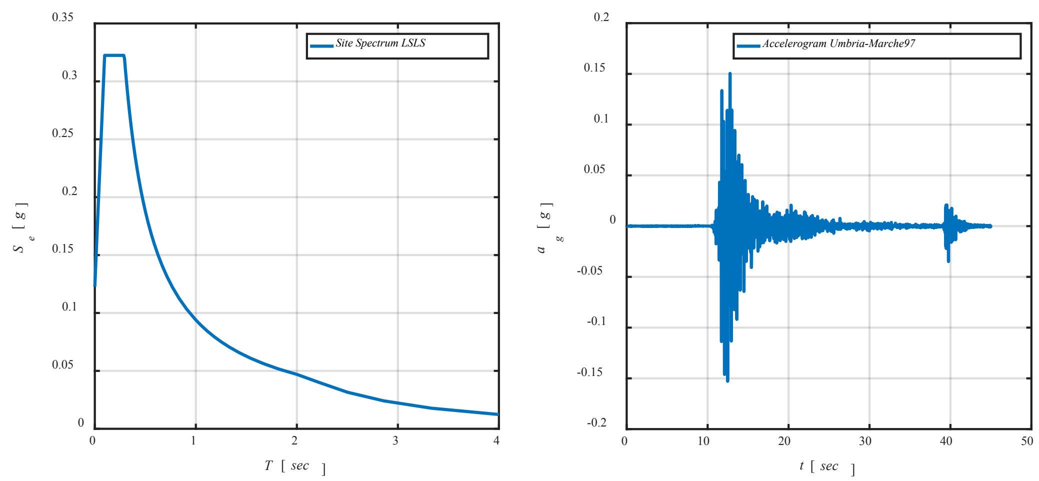


Figure 10: Earthquake information: a) local site spectrum for the LSLS, and b) one of the seven natural accelerograms compatible with the spectrum.

4.5 *Parametric studies to single out the optimal solution*

Assuming that a single cylindrical friction pendulum device was adopted for the case study analyzed, the non-linear dynamic analyses were carried out parametrically in order to single out the optimal solution in terms of input parameters. The parameters whose influence was investigated are the followings (Fig. 11): dynamic friction coefficient μ_d; curvature radius of the sliding surface R_i; and length of the FPD L_i. The range of values for each of the input parameters are listed in Table 2, along with those of the reference solution. The value of the static friction coefficient was assumed equal to $\mu_s = 1\%$ since what substantially affects the analyses is the value μ_d of the dynamic friction.

The output results analyzed were the following: a) the ratio $\sigma_{1Sd,St}/\sigma_{1Rd,st}$ between the maximum vertical stress transferred to the underlying masonry in the static limit state load combination $\sigma_{1Sd,St}$ and the masonry design capacity $\sigma_{1Rd,st}$; b) the ratio $\tau_{2Sd,Eq}/\tau_{2Rd,Eq}$ between the transversal shear stress transferred to the underlying masonry in the LSLS seismic load combination and the masonry design capacity $\tau_{2Rd,Eq}$; c) the ratio $u_{2Sd,Eq,max}/u_{2Rd,Eq}$ between the maximum transversal displacement demand corresponding to the LSLS seismic load combination $u_{2Sd,Eq,max}$ and the relevant device capacity $u_{2Rd,Eq}$; d) the ratio $u_{2Sd,Eq,rdl}/u_{2Rd,Eq}$ between the residual transversal displacement at the end of the earthquake and the relevant device capacity $u_{2Rd,Eq}$. The values of the masonry capacities $\sigma_{1Rd,st}$ and $\tau_{1Rd,Eq}$ were evaluated adopting the minimal values listed in Table C8A.2.1 of the explanatory document annex to the Italian Building Code [31]. Such values were taken for a masonry made of irregularly shaped tuff blocks and assuming safety factors γ_m equal to 3 and 1 for static and seismic conditions, respectively, and Confidence Factor $CF = 1{,}35$ corresponding to a limited knowledge level.

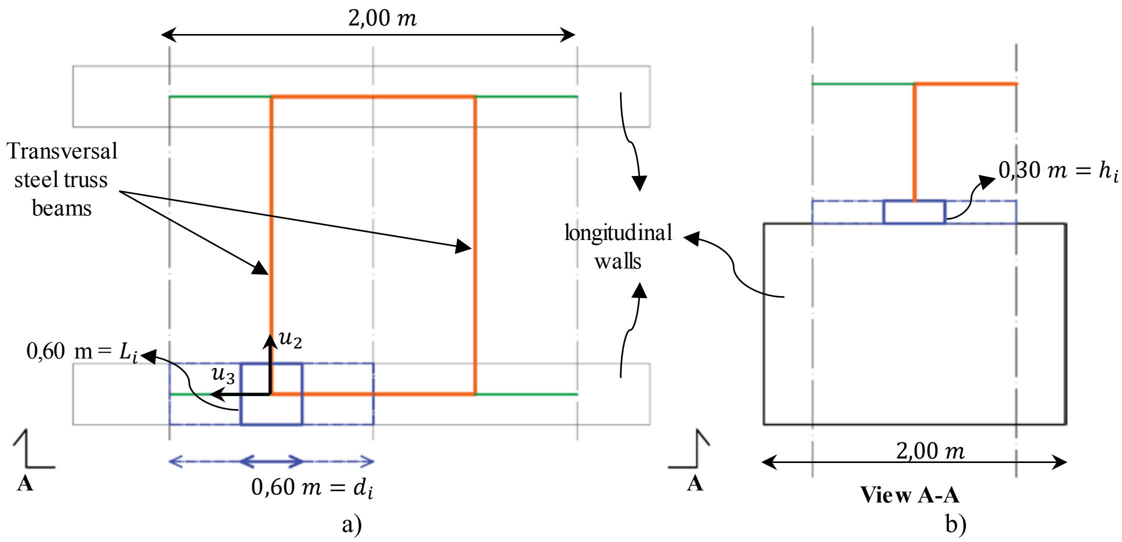


Figure 11: Steel truss deck *Module* adopted for the restoration design of the “Galleria delle volte crollate”: a) plan and b) lateral view. Note that the frictional bearing is here represented by the equivalent prism.

The parametric studies were carried out taking advantage of the *batch file* utility offered by the software SAP2000® and for four earthquakes, that are: 1) the earthquake Marche-Umbria1997 (MU97) (Eq1); 2) the earthquake MU97 scaled by 1,5 (Eq2), 2 (Eq3) and 2,5 (Eq4). The peak ground acceleration ranged from $0{,}157\ g$ to $0{,}38\ g$. The non-linear time history analyses were carried out using a modal analysis based on the load-dependent Ritz vectors method.

Input parameters	μ_d *[%]*	L_i *[cm]*	R_i *[m]*
Reference solution	9,0	1,3	3,5
Range of variation	1,0-18,0	0,6-2,0	2,0-5,0
Optimal solution	18,0	0,6	2,0

Table 2: Values for the input parameters adopted for the parametric studies.

The optimal solution can be identified as the combination of values of the input parameters, *i.e.*, L_i, R_i, and μ_d that yield the simultaneous fulfillment of the following four criteria:

$$\sigma_{1Sd,St}/\sigma_{1Rd,st} \leq 100\ \% \tag{11}$$

$$\tau_{2Sd,Eq}/\tau_{2Rd,Eq} \leq 100\ \% \tag{12}$$

$$u_{2Sd,Eq,max}/u_{2Rd,Eq} \leq 100\ \% \tag{13}$$

$$u_{2Sd,Eq,rdl}/u_{2Rd,Eq} \leq 100\ \% \tag{14}$$

These inequalities will help single out the range of values for the input parameters corresponding to the possible solutions as indicated qualitatively in Fig. 12 below.

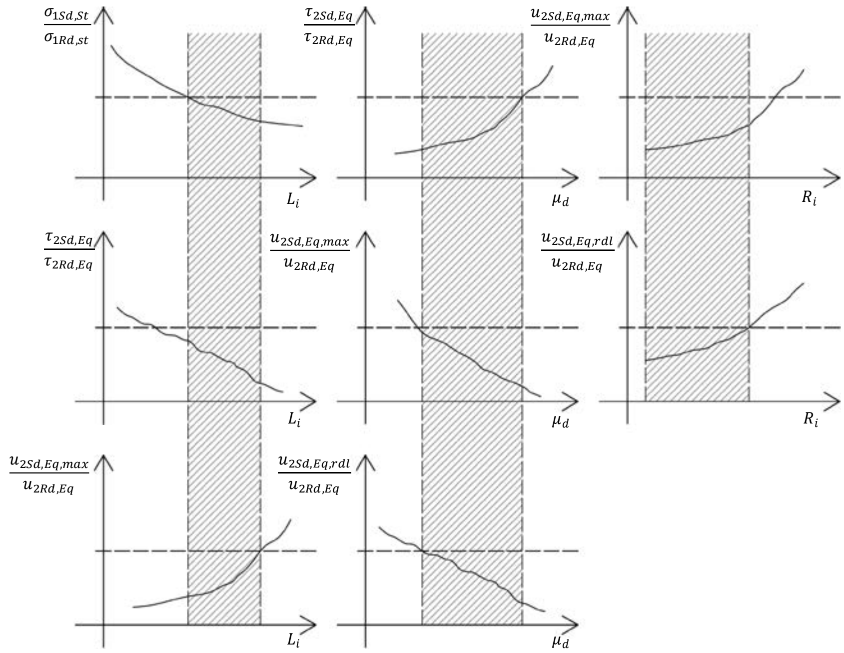


Figure 12: Optimization procedure based on the results of the parametric studies: qualitative expected trends.

As to the case study of the *Galleria delle Volte Crollate*, the results of the parametric studies are represented in the next figures from Fig. 13 to Fig. 15. Note that the several curves plotted refer not only to the earthquake UM97 (Eq1) but also to the scaled-up earthquakes (Eq2 to Eq4), even if these latter were taken into consideration only for comparison purposes. In case all the seven earthquakes were considered, what depicted in Fig. 12 would still apply, except that the average curve would be considered, instead. The trends obtained for the curves corresponding to the parametric study concerning Eq1 are in agreement with what expected. For instance, as to the influence of the friction pendulum curvature radius (Fig. 13), when R_i is increased, the post-slipping stiffness k_{ps} would decrease and both the maximum and the residual displacements would increase accordingly.

As to the influence of the friction coefficient μ_d (Fig. 14), as expected, by increasing it, the tangential stress exerted on the extrados of the masonry in earthquake conditions increases since the Coulomb Friction Law threshold increases in turn. On the contrary, both the maximum $u_{2Sd,Eq,max}$ and residual $u_{2Sd,Eq,rdl}$ transversal displacement decrease.

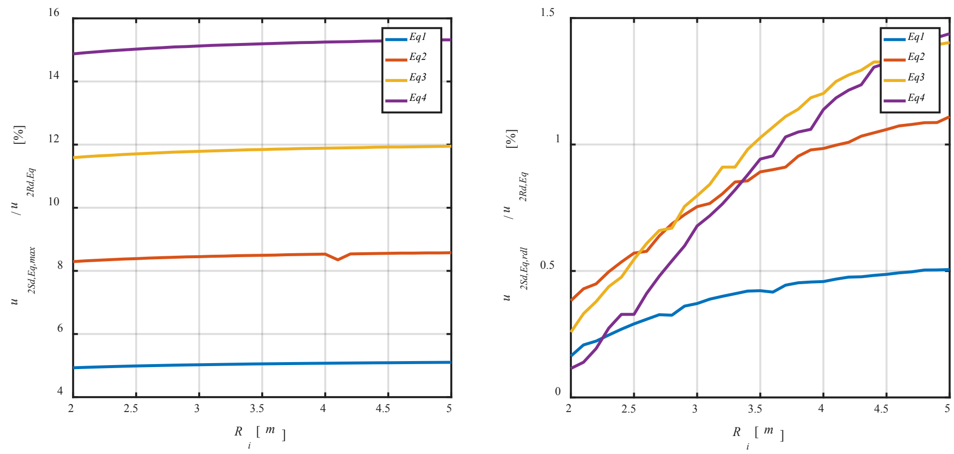


Figure 13: Results of the parametric studies concerning the effect of the FPD curvature radius.

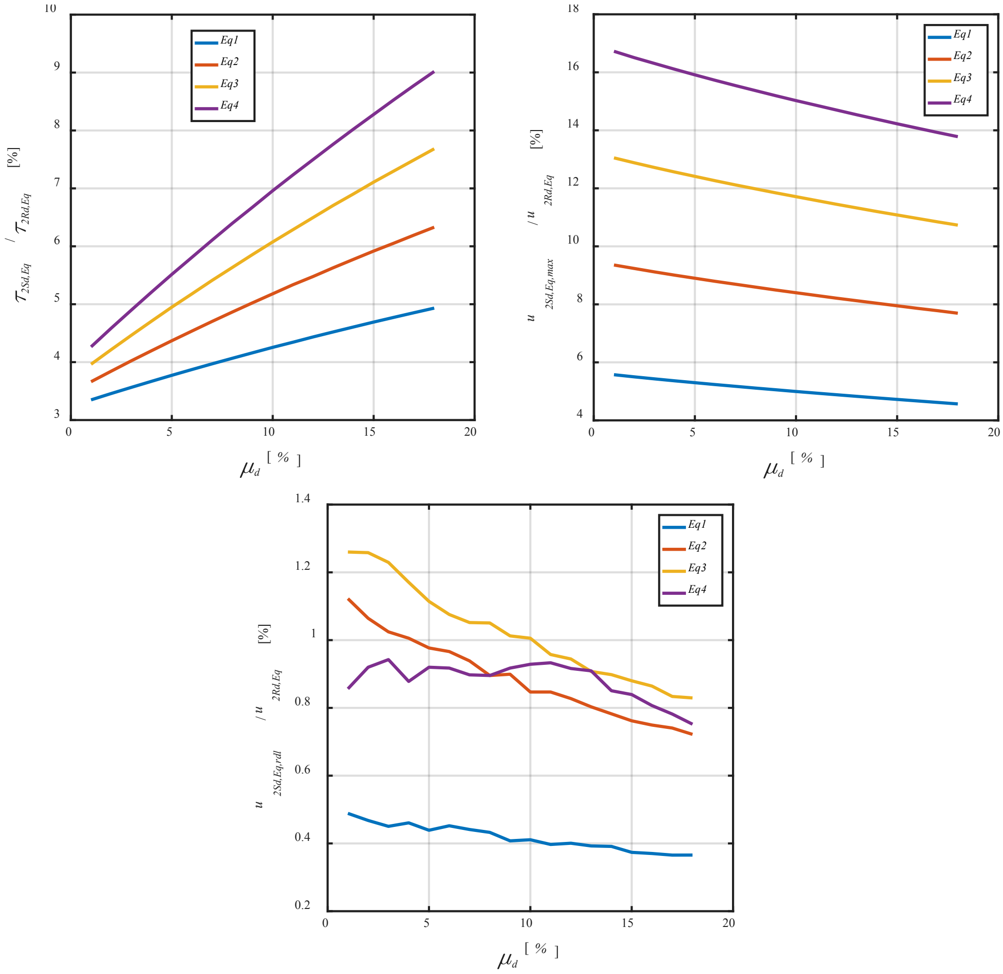


Figure 14: Results of the parametric studies concerning the effect of the FPD friction coefficient.

By increasing the longitudinal extension L_i of the bearing (Fig. 15), both the maximum value of the static vertical pressure $\sigma_{1Sd,St}$ and the maximum value of the seismically induced tangential stress $\tau_{2Sd,Eq}$ exerted on the underlying masonry decrease since the same loads, vertical and horizontal respectively, are spread over a larger surface. On the contrary, both the maximum $u_{2Sd,Eq,max}$ and residual $u_{2Sd,Eq,rdl}$ displacement slightly increase by increasing L_i.

Note that, all the curves referring to the Eq1, that is UM97, are below the value of 100%, for the case study herein analyzed. This is due to a twofold reason: 1) the zone seismicity is relatively low, and 2) the masses were drastically reduced by preserving only the cortical parts of the collapsed masonry blocks. In such cases, the value for each of the input parameters may be based on an economical criterion. We can choose the larger value of μ_d since it will generally imply the use of more common materials – plain PTFE instead of PTFE lubricated by Glass insertions. Likewise, we can choose the shorter value of L_i since it will imply the use of a shorter and less expensive cylindrical bearing. Moreover, we can choose the larger value of R_i since it will mean, for the same value of depth of the bearing, less smoothing work.

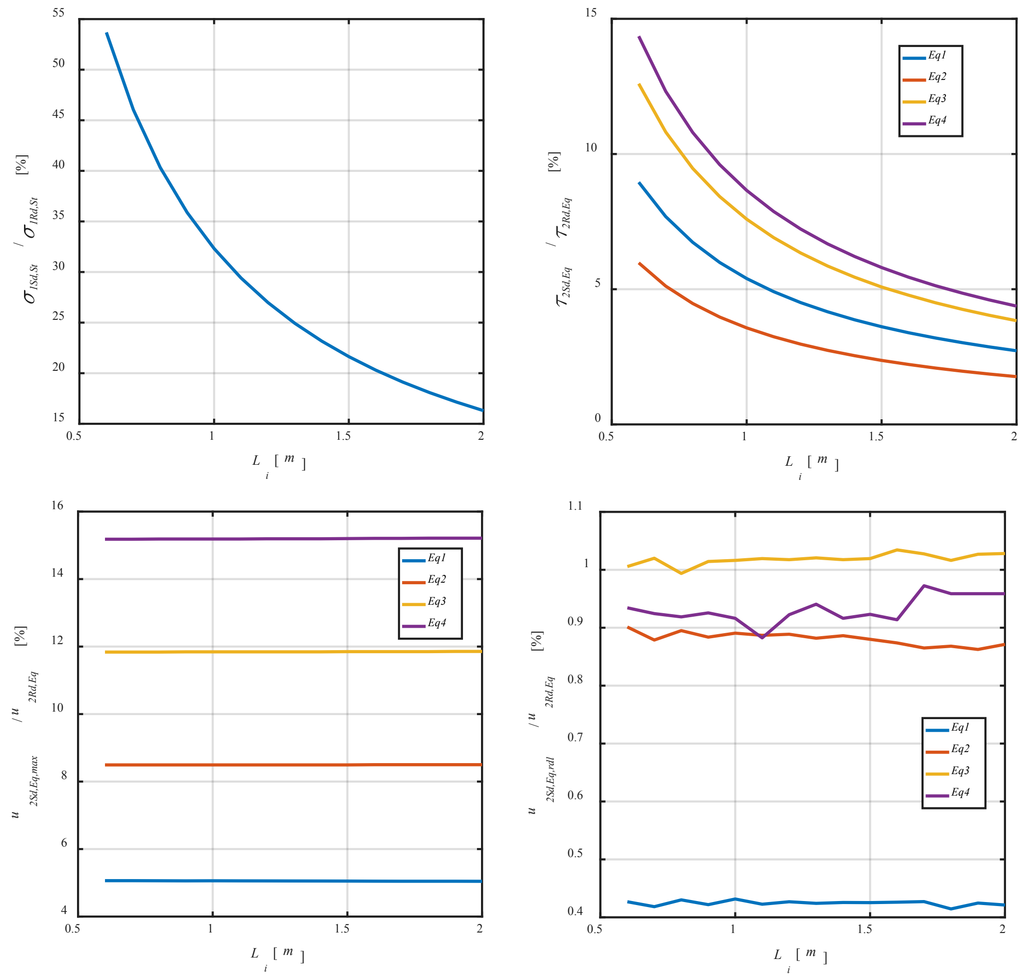


Figure 15: Results of the parametric studies concerning the effect of the FPD length.

5 CONCLUSIONS

The present work dealt with the topic of the reconstruction of collapsed archaeological-valued masonry vaults starting from the case study of the *Galleria delle Volte Crollate*, in Rome.

The possibility to intervene by adopting frictional bearings was explored. Such devices can be either flat, in case of low seismicity zones, or cylindrical, in case of higher seismicity. This is due to the necessity of warranting a suitable displacement re-centering capability. Moreover, the length of such devices can be calibrated on the basis of the maximum vertical stress that the underlying masonry can stand. Likewise, the friction coefficient can be calibrated on the basis of the maximum transversal load that the underlying masonry can bear during an earthquake. After calibrating the friction coefficient, the corresponding couple of materials constituting the kinematic pair can be suitably selected.

The selection of the optimal combination of values for the input parameters was obtained by means of parametric studies. As first approximation, the movement of the underlying walls and the steel truss deck were considered uncoupled, but such approximation will be removed in further developments.

Further parametric studies will be carried out in order to deduce more information that may be useful for practitioners interested in applying the solution strategies herein described. Moreover, the possibility to come up with a more formally elegant and compact optimization strategy will be investigated.

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