Seismic Improvement of Monumental Churches with Domes

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SUMMARY:
The seismic vulnerability of churches, even for events of low intensity, is often conditioned by the response of individual structural elements. The elements placed in the top of the structure such as domes, lanterns, tiburios, etc.. are particularly vulnerable. The mechanism is well known among the possible damages caused by a seismic event, and it is included in the list of structural vulnerabilities of the churches. Recent examples of the mechanism and its devastating effects on the building are visible in all the major earthquakes: L'Aquila (Santa Maria del Suffragio Church); Haiti (Presidential Palace); Christchurch (Historical Building). A new solution using new technologies (isolators and dampers) for seismic improvement of buildings is proposed in this paper. Using this technologies the dome is changed into a Tuned Mass Damper (TMD). The proposed solution is investigated with regard to the seismic improvement project of the 16th century church of San Nicolò in Carpi, Italy.

Keywords: Monumental Churches, Domes, TMD, Isolation

1. INTRODUCTION

The seismic vulnerability of churches, even for events of low intensity, is often conditioned by the response of individual structural elements. The elements placed in the top of the structure such as domes, lanterns, tiburios, etc.. are particularly vulnerable. The mechanism is well known among the possible damages caused by a seismic event, and it is included in the list of structural vulnerabilities of the churches, in italian guidelines on seismic risk reduction in historical buildings.
The hazardousness of such a mechanism of damage is made worse by the fact that a collapse of parts of the structure above the naves of the church also causes damage to the underlying structure as a result of the collapse. Recent examples of the mechanism and its devastating effects on the building are visible in all the major earthquakes: L'Aquila (Santa Maria del Suffragio Church); Haiti (Presidential Palace); Christchurch (Historical Building).

L'Aquila (Italy) - 2009

Port Au Prince (Haiti) - 2010

Christchurch (New Zealand) - 2011

Figure 1. Examples of the mechanism and its devastating effects in recent Earthquakes

To avoid this damage mechanism, the actions proposed until now consist of the structural reinforcement of domes and lanterns. These types of intervention, while making the domes monolithic, not only don't improve, but even deteriorate the stability conditions by stiffening the upper part of the structure and moving the problem to the columns that support these elements.
A proposed alternative solution to reduce the vulnerability of these structures is to separate the two parts of the structure by inserting an isolation system that dynamically separates the upper parts (domes, lantern, tiburios) from the rest of the structure.
Even more interesting can be investigating the possibility to change the dome into a Tuned Mass Damper system for the underlying structure. By coupling appropriately sized dampers to the isolation system, the benefit of the intervention could be extended to the whole structure.
This procedure has been proposed and investigated for the seismic improvement of the church of San Nicolò in Carpi, where a seismic improvement intervention conducted in the past to create a rigid floor at roof level, was already performed.

1.1. Tuned Mass Dampers

Tuned Mass Dampers (TMD) are usually mass-spring-dashpot systems that are "tuned" to be in resonance with a particular mode of the structure on which they are installed. These systems are capable of significantly reducing the dynamic response of a structure, yet their components are simple. In its simplest form, a TMD only requires the assembly of a mass, a spring and a viscous damper at a given point of the structure, with no need for an external power source or sophisticated hardware.
In the case of the domes, we can take advantage of the following favourable conditions:

- The mass of the TMD is the mass of the dome; the mass is precisely identified, there is no significant change in mass due to applied loads;
- The dome is generally in the upper part of the church and large displacements are allowed in every direction;

The purpose of adding a TMD is to bring the resonant peak of the amplitude of displacements or accelerations to its lowest possible value. The problem is to find an "optimum tuning" of the TMD.
Simple theories have been developed for undamped linear single-degree-of-freedom structures subject to sinusoidal dynamic forces, but they have to be improved in order to consider that civil engineering structures are multi-degree-of-freedom systems; they may experience nonlinear behavior and earthquake action is random excitation and not sinusoidal. Several optimum tuning conditions have been developed by researchers, based on different optimization criteria and loading cases; details can be found in Sadek, F., Mohraz, B, Taylor, A., and Chung, R., A (1997). Anyway, for small mass ratios, the difference between different tuning conditions are relatively small. Since a TMD can only be tuned to a single structural frequency, it is expected that the effectiveness of a TMD is the greatest when the structure vibrates mainly in a predominant mode.

2. SEISMIC VULNERABILITY REDUCTION

Looking at the effects of recent earthquakes on historical buildings, the fragility of domes, tiburios and vaulted elements becomes evident. Not all domes have the same vulnerability, as they have different geometry, dimensional ratios, materials. The domes, especially in past centuries, has always played a role of particular architectural importance, as witnessed by the world renown Brunelleschi's Dome (S. Maria del Fiore, Florence), Michelangelo's Dome (S. Pietro, Rome), the White House (Washington 1792), Saint Sofia (Istanbul 537), whose dome was destroyed after the 558 earthquake and restored after subsequent earthquakes. Even if not so famous, there are many churches in which the dome and the underlying tamburo are significantly tall. For these particular structural configurations, the dynamic response of the dome play an important role in the overall response of the building. Looking at the dynamic behavior of this type of buildings it can be observed that the main modal shapes often involve the dome, which is usually set on columns that can be seriously damaged by the movement of the dome, when the dome itself is not damaged.
The use of modern technologies for seismic protection, such as base isolation, can separate the motion of the dome from the motion of the church. Therefore, isolation firstly allows to protect the dome from damage by reducing the seismic action transmitted to it. Furthermore, the considerable mass of the dome can be assumed to use the dome itself as TMD for the reduction of the seismic vulnerability of the underlying structure. The optimal solution is a compromise between the safety of the dome, with reducing seismic acceleration, and the improvement achieved by the underlying structure.
As each historic building is different in configuration and proportion, a valid solution that fits all cases doesn't exist. Therefore, the study relating to a specific structure is reported in this paper.

3. SAN NICOLO' CHURCH

The church of San Nicolò is one of the main churches of Carpi, a medieval village not far from Modena, of prehistoric origins. Starting from the fourteenth century Carpi was the seat of the Signoria of Pio, that began construction of the church, that will become part of the domains of the Signoria of Este from the sixteenth century.

3.1. Description of the structure

The structure of the church is typical of similar buildings dating back to the same period. Experts believe that the present church originates from the completion of a first part (Phase I construction) with the addition of naves attributed to Baldassarre Peruzzi (Phase II construction). Construction of the church began in 1494 and was completed in 1522; the church was consecrated April 26, 1522.

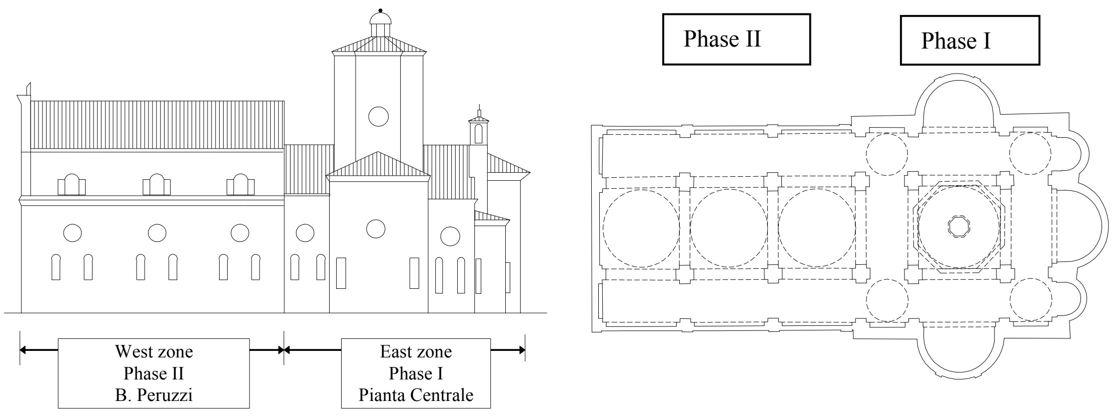


Figure 2. Construction phases of the Church of S. Nicolò

The vertical structure of the church is built with masonry walls and the roof is made of wooden beams. On the basis of endoscopic tests performed in 1998, it was found that the perimeter wall is generally continuous and compact, for the presence of limited void. The texture within the thickness of the wall is more disordered than the surface, with irregular pieces of brick embedded in the mortar.
The columns have nearly constant cross sections throughout the church. From sonic thermographic tests a fairly homogeneous texture of the columns and a poor quality of masonry results, with a small presence of voids and other internal defects.

3.2. Past earthquakes

The available documentation provides information about the damages in the church as a result of the earthquakes of 1832, 1987 and 1996, thus allowing to identify the main vulnerabilities of the structure (before of seismic retrofitting). Omitting what happened as a result of oldest earthquakes, we will focus on 1996 earthquake. This earthquake damaged the church significantly. The works for the seismic upgrade started in 1998 and ended in May 2003. The intervention was designed and implemented, with the aim of creating a "box-like behavior" of the building, through the insertion of rigid connections at the top the walls, at the roof level. The available documents show that the seismic upgrade leads to a reduction of the displacements, without any indication about the seismic vulnerability reduction achieved.

4. CONDITION ASSESSMENT

Comprehensive investigation of the vulnerability of the building have been conducted. Tests on site ground, on materials and on structures have been performed (Sonda, D. et al. 2006 - 2009),. The following considerations are based on the italian "Guidelines for the evaluation and reduction of seismic risk of cultural heritage" (2008).

4.1 Required Performance Objectives

The Guidelines indicate that the nominal life of cultural heritage should be very long, wishing to ensure the preservation in time also with respect to seismic activities with a long recurrence interval (high intensity); however, this would lead to heavy conditions for seismic assessment and invasive retrofit interventions would be necessary. In line with the possibility of simply upgrades, the project can refer to a shorter nominal life. This reduced rated life (even less than 50 years) will allow in any case of certifying the safety of a less invasive intervention, as this will protect the construction, in probabilistic terms, for a smaller number of years. At the end of nominal life, a new assessment should be performed, and consequently new interventions may be necessary, but at that time it will be possible to take advantage of progress in knowledge and technology, in terms of knowledge of seismic hazard, ability to assess the vulnerability of the construction and availability of less invasive retrofit techniques. Therefore, the period between an improvement intervention and the next can be used as the nominal Life. The performance objective for this structure was Life Safety Limit State (SLV) for the design earthquake with a recurrence interval of 712 years (10% probability of exceedence in 75 years). Consequently the design referred to the following Peak Ground Acceleration: $PGA = a_g \cdot S = 0{,}180 \times 1{,}428 = 0{,}2570$ g.

5. MATHEMATICAL MODELS

5.1 Overview

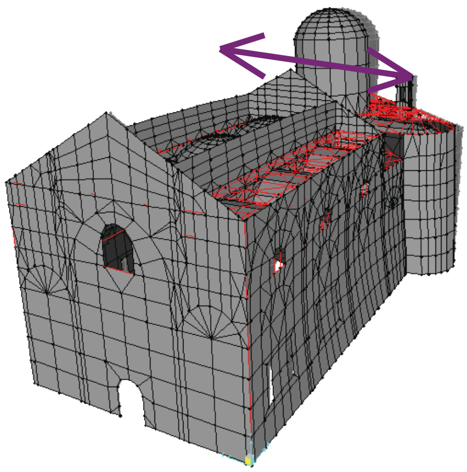
Computer program SAP (CSI 2008) was used to prepare mathematical models of the structure. All pertinent mass and stiffness components were incorporated in the models. Three-dimensional models were used in analyses.

5.2 Calibration of Analysis Models

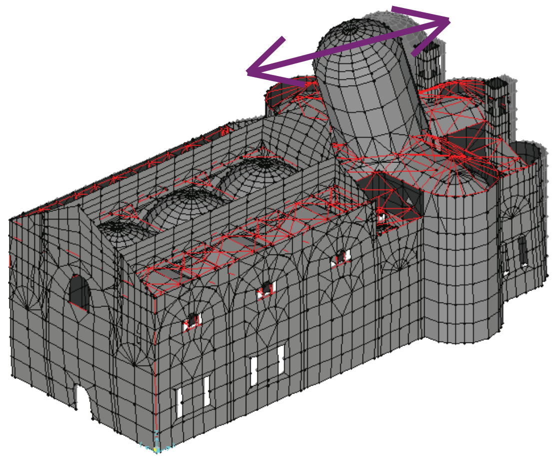
In order to make a direct comparison between the dynamic behavior of the mathematical model and the real structure, the modal analysis was performer by using an Elastic Modulus of 2250 N/mm^2, corresponding to the initial tangent modulus of elasticity measured in situ with double flat jack for a stress range of 0.30 to 0.60 N/mm^2.

Field tests were conducted to determine the dynamic properties of the structure. This data was then used to verify the accuracy of the mathematical model of the building and to design the building upgrade. Field tests consisted of ambient vibration surveys, the structure was subjected to low amplitude natural vibrations and the acceleration data was collected using accelerometers. Instrumentations were strategically placed, so as to compute the transfer function and mode shape.

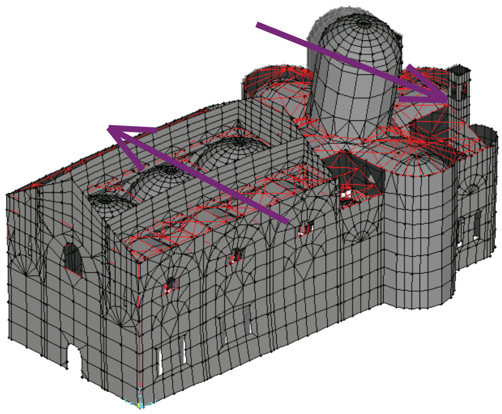
Figure 3 shows that the analytical model closely predicts the fundamental modes of the core and the upper arches in the principal directions



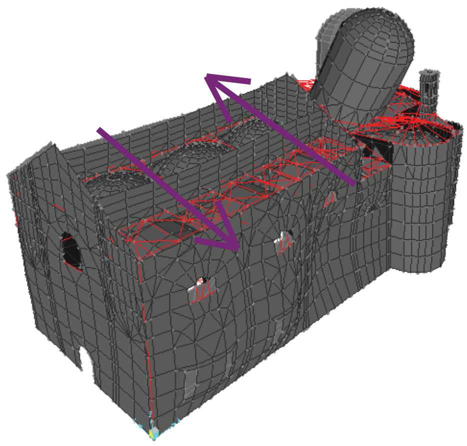
1th mode : trasversal direction; $f = 1{,}77$ Hz
Measured frequence: $f_m = 1{,}98$ Hz - $\Delta = -10\%$



3rd mode: longitudinal direction; $f = 2{,}59$ Hz
Measured frequence: $f_m = 2{,}59$ Hz - $\Delta = 0.0\%$



4th mode : transversal direction, torsional in plant;
$f = 2{,}90$ Hz
Measured frequence: $f_m = 2{,}71$ Hz - $\Delta = +7{,}0\%$

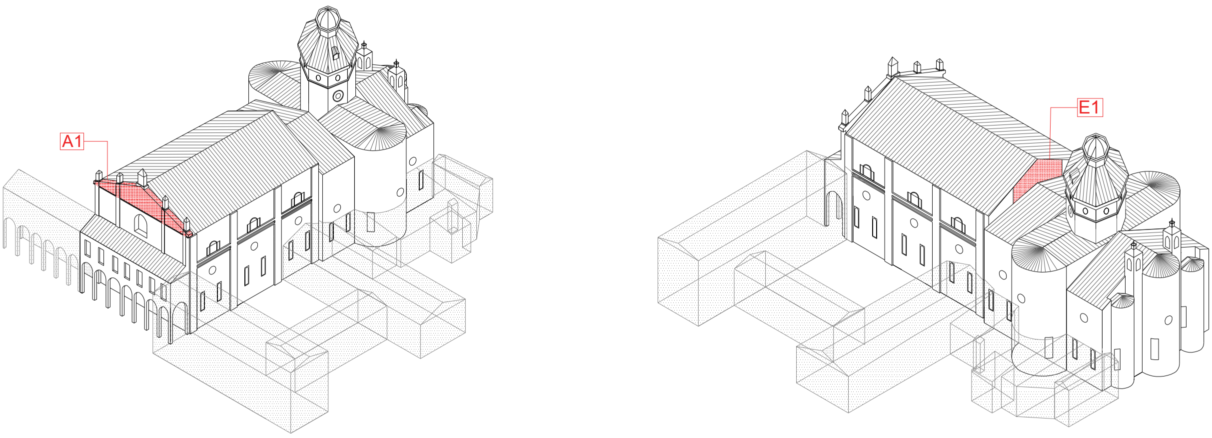


6th mode : transversal direction, full wave in plant;
$f = 3{,}29$ Hz
Measured frequence: $f_m = 3{,}54$ Hz - $\Delta = -7{,}0\%$

Figure 3. Modal shapes and frequencies.

6. PERFORMANCE OF THE EXISTING BUILING

The structural strengthening interventions performed in the past have produced a box-like structural behavior. The overall vulnerability of the structure and the individual damage mechanisms were quantified in terms of ratio between the acceleration of activation of the mechanism and the maximum code-level acceleration ($\alpha = PGA_C$ / PGA_D). The ratios obtained show that the local mechanisms of overturning of limited portions of the building have lower acceleration of activation. The mechanisms with the lowest acceleration of activation are shown in the figure 4.



$\alpha = 0{,}360$ $\alpha = 0{,}241$

Figure 4. Local damage mechanisms with lower acceleration of activation

7. SEISMIC RETROFIT

Looking at the building as a whole, it's very difficult to quantify the vulnerability of the tiburium placed over the transept, which simultaneously is the first element to be excited by vibration of the structure. The top of the columns, placed in support of the tiburio, was damaged in every past earthquake. This concern led to believe that, for the reduction of seismic vulnerability of the building, particular attention should be paid to this element.

7.1 Proposed retrofit solution

A seismic retrofit was investigated to both increase capacity and to reduce demand. A tuned mass damper (TMD) approach was selected to reduce seismic demand. This choice was based on structural, aesthetics, and financial considerations. For this type of stiff and tall structure, the conventional retrofits, such as strengthening or adding external pos-tensioning tendons, would have altered the buildings appearance and hence were not feasible.
Before cutting the tiburio approximately at its base, metallic plates will be inserted inside the mortar joints in 3 courses of bricks, above and below the cut. Niches will then be created in the wall, in each of the vertices of the octagon base, to accommodate devices.
Stiffness and damping will be supplied by a system of eight rubber bearings and eight fluid viscous dampers respectively. As a result, most of seismic motion will be taken up by the TMD, thus reducing drifts and seismic demand of the structure. Based on the mass of the existing dome, a TMD with a mass ratio of 7.1% was obtained. The TMD properties were computed based on the procedure developed to obtain the minimum absolute acceleration amplitude of the primary structure (Constantinou et al. 1998). The optimal tuning of the TMD yields to the following parameters:

$$\mu = 0.071; \qquad f = \frac{1}{1+\mu} = 0.934 \qquad \xi_{opt} = \frac{c}{c_c} = \sqrt{\frac{3 \cdot \mu}{8 \cdot (1+\mu)^3}} = 0.158$$

In this equation, μ is the ratio of TMD to first mode mass, f is the ratio of TMD to first mode frequencies, and ξ_{opt} is the optimal damping of the TMD. For practical application to a real system, it is necessary to obtain applicable parameters for the TMD, such as the optimal TMD stiffness k and the optimal damping coefficient c. After easy algebraic manipulations the following values can be obtained: $k = 18094$ kN/m; $c = 917.64$ kN·s/m
The TMD system was tuned to the main frequency in the transversal direction of the Church, since in this direction the higher vulnerability was found. Three pairs of spectrum-compatible motions were selected. Recorded earthquake acceleration traces were selected from a database of real records (European Strong-motion Database) and scaled such that their response spectra closely matched the target spectra. Nonlinear analyses were conducted.

8. RESPONSE OF RETROFITTED STRUCTURE

The effect of the retrofit was measured by monitoring some response parameters of the structural model; the parameters reported in Figure 5 were considered:

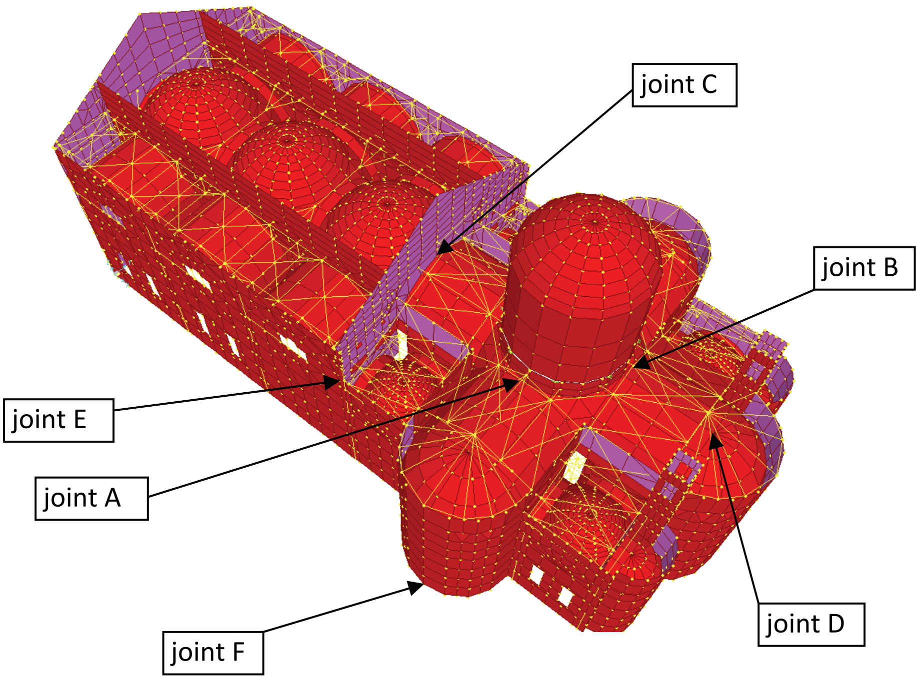
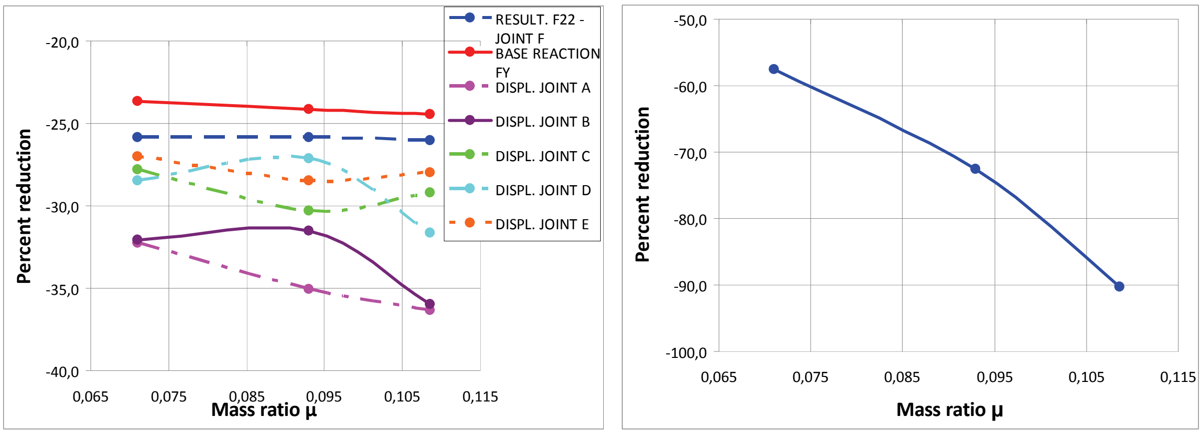


Figure 5. Monitored response parameters

The isolation of the dome results in a significant reduction of relative displacement, and consequently of the forces, between bottom and top of the dome. The numbers show also a significant reduction of the demand in the primary structure, especially in the part of the building that is excited by the fundamental vibration mode.
At a first stage, parameters obtained for single degree of freedom TMD, as defined above, have been used used in the calculations. After, the TMD properties were optimized by conducting sensitivity analyses with different TMD frequencies and damping ratios and evaluating response reductions.
In the first analysis, the TMD properties where calculated with optimal tuning condition for 3 different mass ratios μ, results are presented in Figure 6.



a) b)

Figure 6. a) Primary structure response - Percent reduction vs. Mass ratio μ - b) Dome response - Percent reduction vs. Mass ratio μ

In the second analysis, starting from the properties calculated with optimal tuning condition for the TMD with the existing mass, the stiffness *k* has been varied. The results are presented in Figures 7, 8.

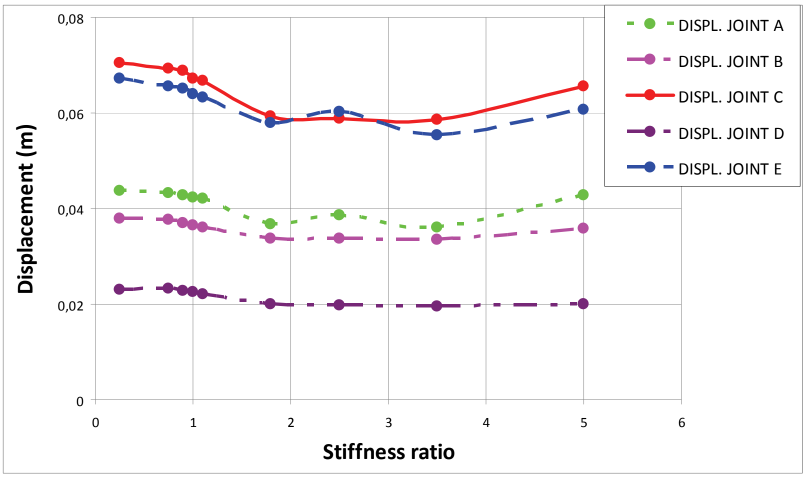
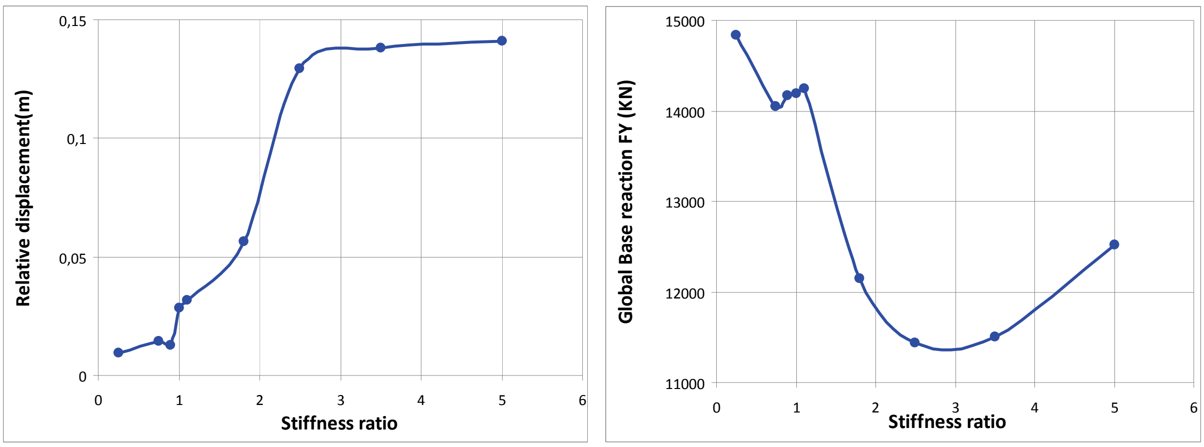


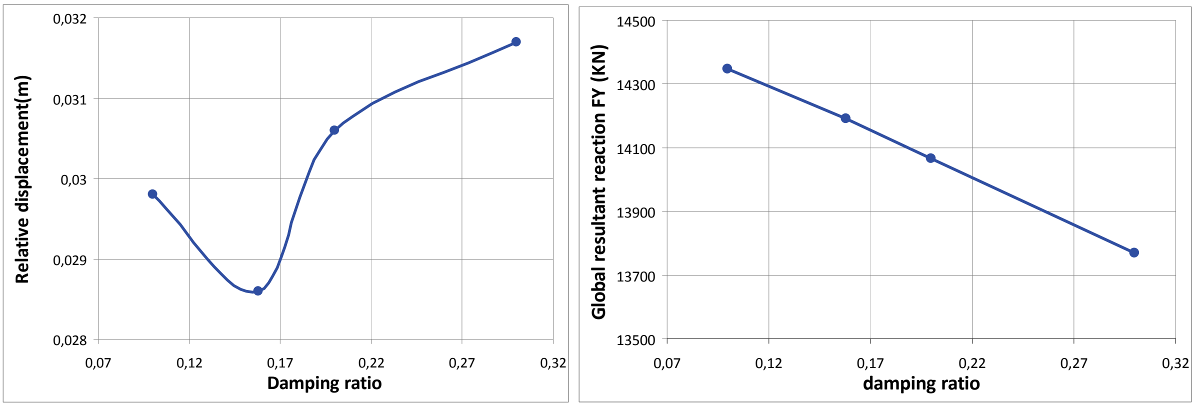
Figure 7. Primary structure response - Joint displacement vs. Stiffness ratio



a) b)

Figure 8. a) Dome response – Relative displacement vs. Stiffness ratio – b) Primary structure response – Global Base Reaction FY vs. Stiffness ratio

In the third analysis, starting from the properties calculated with optimal tuning condition for the TMD with the existing mass, the damping ratio ξ has been varied. The results are presented in Figures 9.



a) b)

Figure 9. a) Dome response – Relative displacement vs. Damping ratio – b) Primary structure response – Global Base Reaction FY vs. Damping ratio

Sensitivity analyses showed that:

- an increase of the dome mass, difficult and costly to realize, doesn't reduce significantly the demand on the primary structure;
- an increase in the damping ratio results in a small reduction of the demand on the primary structure;
- a minimum condition in the demand on the primary structure can be individuated by varying the stiffness of the TMD;
- increasing stiffness and damping results both in a reduction of demand on the primary structure, and in an increase of demand on the dome.

According to the previous results, an optimal solution can be individuated based on the following properties of TMD: no increase in dome mass; damping ratio from the optimal tuning condition; stiffness ratio =1.5. The resultant properties of the TMD are listed below:

$\mu = 0.071$; $k = 1.5*18094 = 27141$ kN/m; $c = 917.64$ kN·s/m

Nonlinear analyses have been performed with the optimal properties, obtained after the sensitivity analyses, and some significant results are presented in Table 8.1 and in Figure 10.

Table 8.1. Existing vs retrofitted with optimized TMD

PARAMETER	EXISTING	W/TMD - OPTIMAL	DIFF. %
Result. Force F22, Joint F [KN]	1307.3	933.5	-28.6
Global Base Reaction FY [KN]	18594	13189	-29.1
Displacement Joint A [m]	0.0626	0.0387	-38.2
Displacement Joint B [m]	0.0536	0.0341	-36.4
Displacement Joint C [m]	0.0932	0.0634	-32.0
Displacement Joint D [m]	0.0316	0.0211	-33.2
Displacement Joint E [m]	0.0877	0.0600	-31.6
Dome Relative Displacement [m]	0.0675	0.0368	-45.5

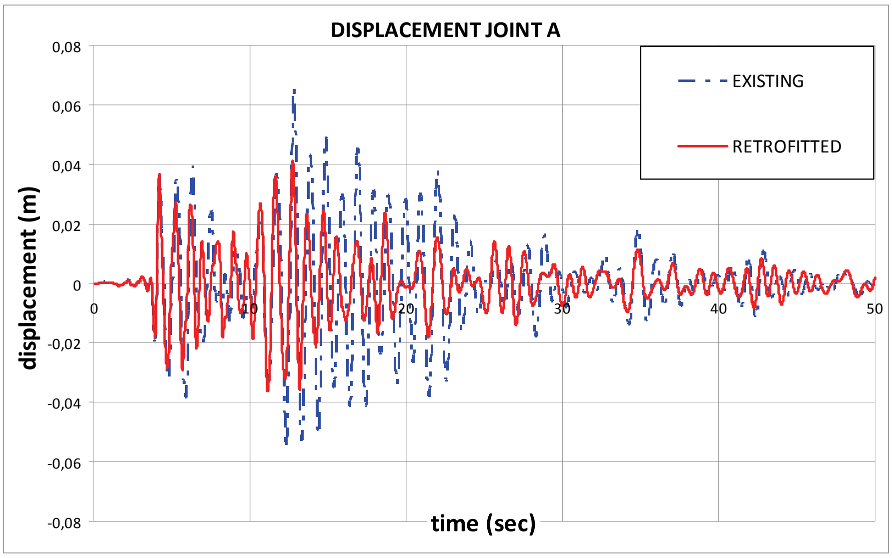


Figure 10. Response of existing and retrofitted structure (displacements joint A).

9. CONCLUSIONS

In the paper a new technique for seismic upgrading of historic buildings with a dome through the dynamic decoupling of the dome from the rest of the structure is proposed and discussed. The "optimal tuning" of the properties of seismic protection devices (isolators and dampers) allowed to turn the dome into a TMD system, that improves the seismic performance of the entire building. The proposed technique is discussed for the application on the church of S. Nicolò in Carpi. The structure presents a tall tiburio with a dome, and has undergone recent major seismic upgrade interventions, that not fit those required by current regulations. The upgrade intervention already performer have tended to produce a set box-like structural behavior of the building. Looking at the building as a whole, the part of which is more difficult to quantify the vulnerability, but which simultaneously turns out to be the first to be excited in the first vibration modes, results to be the tiburio, which is is placed over the transept. A collapse of this part of the building, located in a position overlying the others, would cause damage to the rest of the church very seriously. This concern led to believe that, for the reduction of seismic vulnerability of the building, particular attention should be paid to this element.
The use of the dome as a TMD system can be considered a cost-effective solution to reduce the vulnerability of many historical buildings.

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