Structural Analysis of Historical Constructions, New Delhi 2006
P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.)

Design Aspects in Seismic Isolation of Churches

G. Cuomo, A. De Luca, E. Mele and A. Romano
University of Naples "Federico II", Department of Structural Analysis and Design, Naples, Italy

ABSTRACT: The severe damage of masonry buildings after earthquakes has pointed out the need of studying new seismic protection techniques in order to guarantee appropriate safety level against earthquakes. In historical buildings, like basilica churches, a seismic rehabilitation should have also a minimum impact on the construction, preserving the original structure as far as possible. The Base Isolation System (BIS) balances the opposite requirements of structural safety and architectural preservation. In the light of a previous study conducted by the authors, in this paper an improved optimization process has been applied on some study cases. The structures have been analyzed through FEM, and their seismic response has been evaluated through modal and time history analyses. The attention has been focused to assessing the differences between two models, with and without rigid diaphragm connecting the isolators. The analysis results show that the presence/absence of the rigid floor does not imply variations in terms of accelerations, displacements and forces in the upper part of the construction, but only different values of the normal stress on the isolators for vertical load, due do the different deformability of the upper-structure.

1 INTRODUCTION

Monumental masonry structures usually show sufficient safety level under service load and high durability of materials: the structural efficacy is trivially testified by the fact they have survived for centuries, reaching our days. Seismic actions, on the contrary, are one of the most hazardous events for the architectural heritage, and in particular for masonry churches, as demonstrated by the widespread damage reported in the structural surveys carried out in the aftermath of earthquakes (Doglioni et al. 1994, Proietti 1994, Lagomarsino and Podestà 2004).

Base Isolation System (BIS), widely used for seismic protection of new-designed buildings in earthquake prone countries, today is increasingly being used also for the seismic retrofit of existing structures. The appealing features of application of BIS to monumental building is the possibility of reducing the seismic demand to levels comparable to the superstructure capacity, thus requiring minimum alterations in the original fabric. Furthermore, BIS can be a cost-effective retrofit strategy, as compared to traditional ones, if the same target safety level is fixed as input design parameter.

Remarkable examples of seismic upgrading of historic buildings through BIS, realized in USA, between the end of the 80s and the 90s (Elsesser et al. 1991, De Luca and Mele 1996), have shown that the feasibility of the intervention has to be examined under many aspects which are strictly related to the architectonical, structural and historical characteristics of the construction. Some specific design and construction issues which are related to the application of BIS to historic buildings are: the seismic safety level to be adopted for the upgraded building; the comparison among the BIS solution and other conventional retrofit schemes and the reasons underlying the choice of BIS; the interaction between the strengthening interventions in the upper part and at the foundation level; the sequence of the constructional phases, including the installation (placement and jacketing) of the isolation devices and the definition of the structural details necessary to this purpose.

With specific reference to the churches, some additional issues have to be considered in the feasibility study: above all, the existence of underground volumes, such as crypts, as well as of adjacent constructions, are factors which could make unfeasible such a retrofit strategy. Further, the presence of valuable and precious floor at the base of the structure can undermine a plain application of base isolation, since in current design practice a r.c. slab at the isolation level is introduced. Therefore it seems particularly important to assess the effect of either inserting or not a rigid floor connecting the isolators, at the base of the structure.

Concerning the isolation system design, some special aspects should also be taken into account in the application to retrofit of existing structures: in fact the upper structure configuration and properties are given, and it is not viable to modify the structure layout for optimising the isolation system, as usually can be done in the design of a new base isolated building. For this reason the locations of isolators at the base of the buildings is quite restrained, thus some difficulties in reaching the target isolation period can arise, particularly for light-weight structures.

The presence of slender parts protruding from the roof of the main building, such as of bell tower in the case of churches, characterizes a structural configuration which needs a special discussion (Gulkan 1993). The dynamics of these structure configurations is characterized by an amplified response of the tower which acts as a secondary system; therefore the seismic damage caused by past earthquake was frequently concentrated just in the tower itself, with the consequence that the tower's period is further lengthened and the seismic vulnerability increased. In addition, and more important from the isolation design point of view, the large overturning moments transferred to the basement of the buildings need a wide footprint area in order to be properly distributed to the isolation system, thus avoiding possible uplift of the single isolators.

In this paper two church case studies are considered for discussing some specific design problems which came into picture in the retrofit through BIS; in particular the design procedure for the isolation system, which takes into account the Italy code provisions (OPCM 3431), is described and applied to the two case studies; then, the dynamic performance of the structures is assessed both in terms of global behaviour and in terms of stress distribution among isolators.

2 ISOLATION SYSTEM DESIGN

It is well known that in the design process of an isolation system, the best balance between two conflicting requirements, i.e. maximum filtering of the ground motion and acceptable displacements at the structure base, must be achieved. For this aim, target isolation periods in the range of 2.5-3 s, and damping values (expressed in terms of equivalent damping ratio ξ) between 10% and 20%, are usually selected. However a number of additional design parameters have to be properly adjusted in order to gain also a satisfactory performance both of the global isolated structure and of the single isolator units. In the following, the major steps of the design procedure for the isolation system of existing structures is illustrated; High Damping Rubber Bearings (HDRBs), which combine the isolation, damping and load-support functions, are considered as isolation units.

The basic input of the design process is the upper-structure layout, i.e. mass and plan position of centroid G_{mass}; the locations of isolators are grossly defined at the intersections of transversal and longitudinal walls in the church base plan; consequently, the vertical load N_i acting on each isolator is evaluated.

Tentative values of the target isolation period T_{is}, and of the design vertical stress σ_v for the devices, are set respectively equal to 3 s and 6 MPa. The stiffness of the isolation system K_{tot} can be derived by adopting a SDOF approximation, while the diameters of the isolators D_i can be derived as a function of the design vertical stress. Selecting a rubber compound, the value of the shear modulus G is fixed; (soft rubber, G=0.4 MPa, normal rubber G=0.8 MPa and hard rubber G=1.4 MPa).

By fixing an unique value of the first shape factor S_1 (equal to 20) for all isolators and of the second shape factor S_2 (equal to 3), the steel thicknesses t^i_i and t^i_r are computed for the various diameters. However, homogenization of the device total height is necessary for simplifying construction aspects; thus by homogenizing the total heights of the isolators, minimum thickness values of steel shims and rubber layers are obtained and adopted for all devices to respect the limitations for S_1 and S_2 ($S_1 \geq 20$; $S_2 \geq 3$)

The effective values of S_1, S_2, K_{Hisol}, K_{Htot}, T_{is} and the position of the stiffness centroid C_{stiff} are finally computed and the eccentricity between the mass and the stiffness centroids can be evaluate; if the eccentricity is significant, the values of G, T_{is} or σ_v need to be adjusted, otherwise the procedure ends and the specific checks on the isolators, as prescribed by the code provisions, can be performed.

3 ISOLATORS CHECKS

According to the recent seismic code enforced in Italy (OPCM 3431), HDRBs should satisfy the following checks:

Maximum shear deformation in the isolators

$$\gamma_t = \gamma_c + \gamma_s + \gamma_\alpha < 5 \quad (1)$$

where γ_c is the shear deformation produced by the axial compression, γ_s the shear deformation induced by the seismic displacement including torsional effects and γ_α the shear deformation due to the angular rotation. The shear deformation values are computed as specified in the OPCM 3431, at § 10.2.

Elastomer-steel bonding

$$\gamma_S \leq \frac{\gamma^*}{1{,}5} \leq 2 \quad (2)$$

where γ_s is the maximum value of the shear deformation, and γ^* is obtained through experimental tests as indicated in OPCM 3431 at § 10.B.1.

Stresses in the steel plates

$$\sigma_S = \frac{1.3 \cdot V(t_1 + t_2)}{A_r t_S} < f_{yk} \quad (3)$$

where V is the load acting on a single isolator, t_1 and t_2 the thickness of the 2 rubber layers in contact with the plate, A_r the effective reduced area, t_s the thickness of the plate and f_{yk} the characteristic yield strength.

Critical load

$$V_{cr} = \frac{G\ A_r S_1 D}{t_r} > 2V \quad (4)$$

where V_{cr} is the critical load on a single isolator, G the shear modulus, A_r the effective reduced area, S_1 the first shape factor, D the diameter of the isolator, t_r the thickness of the rubber layer and V the load acting on a single isolator.

Rollout

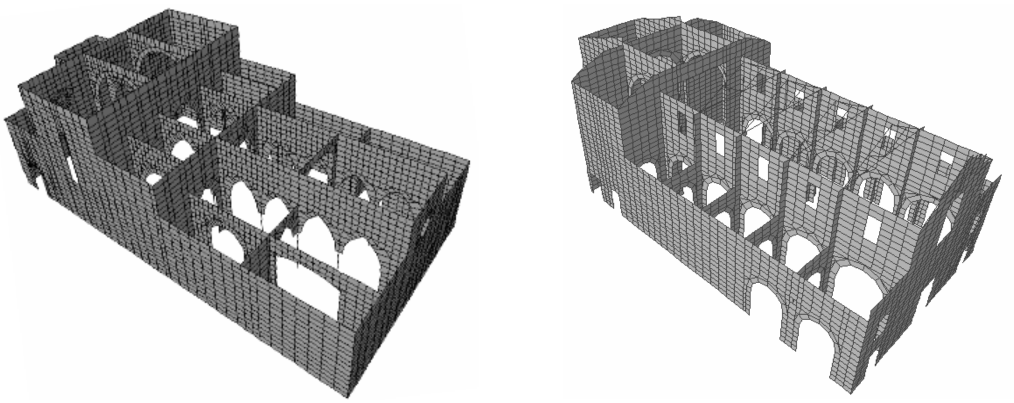
It is a critical check for dowel type isolators (Naeim and Kelly 1999). This check is not contemplated by the OPCM 3431, where the device to upper structure connections are always bolted; however it provides a criterion for limiting the peak tension stress in the rubber induced by the horizontal displacement.

$$\frac{\delta_{\max}}{D} = \frac{V}{V + K_H h} \tag{5}$$

where δ_{max}/D is the maximum displacement normalized to the isolator diameter, V the vertical load, K_H the horizontal stiffness (equal to GA/t_r) and h the total height of the isolator.

4 CHURCH CASE STUDIES

The buildings considered in this study are the “S. Ippolisto” (SI) and “S. Giovanni a mare” (SGMR) churches, both located in Southern Italy. In Fig. 1a and 1b, the 3D FE models of the two buildings are reported.



(a) (b)

Figure 1: Study cases: (a) SGMR; (b) SI: 3D models.

For the two churches, an isolation system, consisting of HDRBs and sliders, has been designed in order to shift the structure period to 3 s; the iterative procedure has been applied until the isolation period was equal the to target value and the mass and stiffness centroids were almost coincident (Fig. 2).

The design procedure has shown different crucial points in the application to the two case studies, characterised by a quite different weight; for the SI church, with a weight of 54830 kN, the isolation target period has been easily achieved, whereas the check for critical load of the single isolators has been the main problem; on the contrary, in the SGMR church, with a much smaller weight (36850kN), all isolator checks were easily satisfied, but the isolation target period was really hard to get, also with small rubber shear modulus (G=0.4MPa).

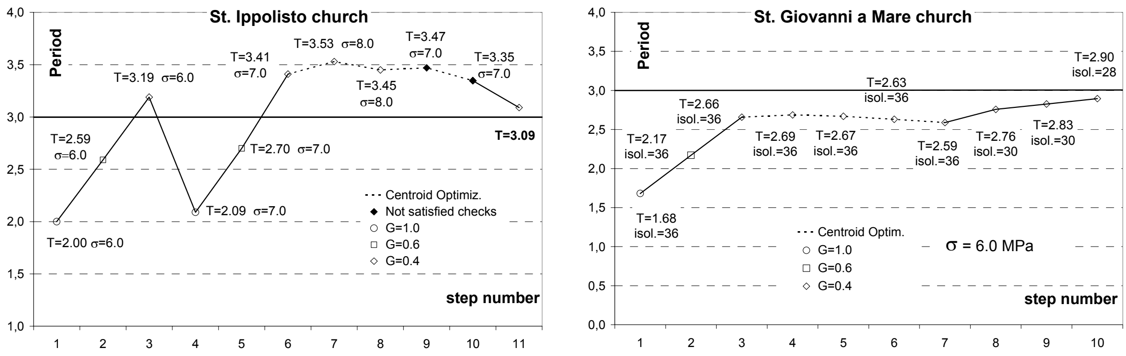
For this reason sliding devices have been placed under some interior walls of the church; the multidirectional teflon sliding devices, constituted by a stainless steel plate fixed to the superstructure and a teflon disk fixed to the substructure, allow any relative translation among the superstructure and the substructure. These devices only carry the vertical loads and do not add lateral stiffness, thus not affecting the isolation period.

These considerations are better shown in Fig. 2a and 2b, where the isolation period is plotted vs. the design iteration number; each step is characterised by a design vertical stress σ, a value of the isolation period T_{is}, a value of the shear module G, and a plan configuration of HDRBs. The target T_{is} equal to 3 s (horizontal line) has been traced as the reference value.

The final HDRBs configuration in the SI church (Fig. 3a) consists of 28 isolators and 5 sliders. It was obtained an isolation period of 3.09 s, with devices ϕ 650 (24 isolators) and ϕ 750 (4 isolators); for both isolator diameters: single rubber layer thickness t_i is equal to 6.5 mm; total rubber thickness t_r is equal to 169 mm.

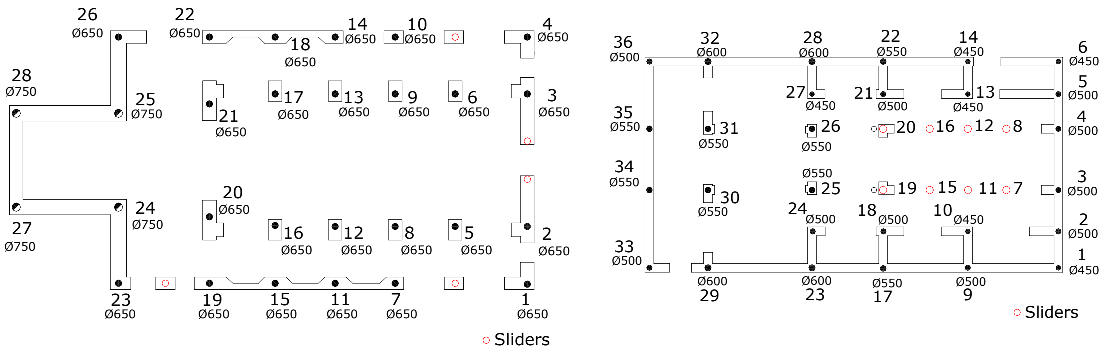
In the isolation system of the SGMR church (Fig. 3b), 28 HDRBs and 4 sliders have been used. The isolation period is 2.90 s and the HDRB diameters are: ϕ 450 (2 isolators); ϕ 500 (6 isolators); ϕ 550 (12 isolators); ϕ 600 (4 isolators); ϕ 650 (4 isolators). Single and total rubber thickness are respectively t_i=5.5 mm and t_r=154 mm.

The thickness of the internal steel plates is assumed, in all cases, equal to 2mm.
In Table 1 the checks for the isolator in the SI churches are reported.



(a) (b)

Figure 2 : Application of the design procedure: iterations: (a) SI; (b) SGMR.



(a) SI – Step 11 (b) SGMR – Step 10

Figure 3 : Placing of the isolators and sliders: (a) SI; (b) SGMR.

Table 1: Checks on SI isolators.

Isolator No.	D	Max shear strain				Bonding	Steel stress	Critical load			Rollout	
	[mm]						[MPa]	[kN]			[mm]	
		γ_c	γ_s	γ_a	γ_t < 5	γ_s < 2	$\sigma_s < 275$	$V_{cr.}$	V	V_{cr}/V > 2	d_{Emax}	$1.5d_E$
											$d_{Emax} < 1.5d_E$	
3	650	2.42	1.33	0.0043	3.75	1.33	136,1	6.9E+03	3.0E+03	2.32	231.9	614.7
24	750	1.95	1.27	0.0031	3.22	1.27	126,8	1.4E+04	4.2E+03	3.31	220.5	711.4

5 EFFECT OF THE RIGID FLOOR AT THE ISOLATION PLAN

In this section, the behaviour of the isolated churches in presence or absence of the rigid diaphragm at the isolation level is discussed in terms of global behaviour (modal shapes, vibration periods and mass participating factors) as well as in terms local isolator demands (stress distribution among isolators).

The analyses carried out for the two churches (both design spectrum analyses and time history analyses with acceleration records) have shown no remarkable differences in terms of global behaviour between the models with/without rigid floor. As expected, the fundamental period of the structure slightly decreases as the rigid slab at the isolation level is introduced; in this case, also larger mass participating factors than in the case of absent rigid floor have been obtained. No coupling effect arise as a consequence of the absence of rigid floor at isolation level and almost no difference has been detected in the displacements at the base and in the superstructure in the two models.

The distribution of the compression stress on the devices are almost similar in the models with or without rigid slab, but in some cases the stress values are quite different; in particular, as shown in Fig. 4, the differences between stress values in the two models (with/without rigid floor) occur (1) due to the only vertical load condition, and (2) for isolators placed under masonry macro-elements characterized by large opening ratio. In Fig. 5 the isolators where the maximum differences between stress values in the two models are shown together with the relevant macroelements.

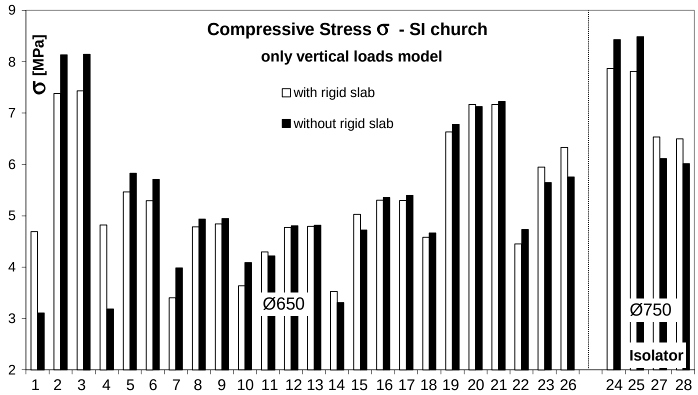


Figure 4. Compressive due to vertical load.

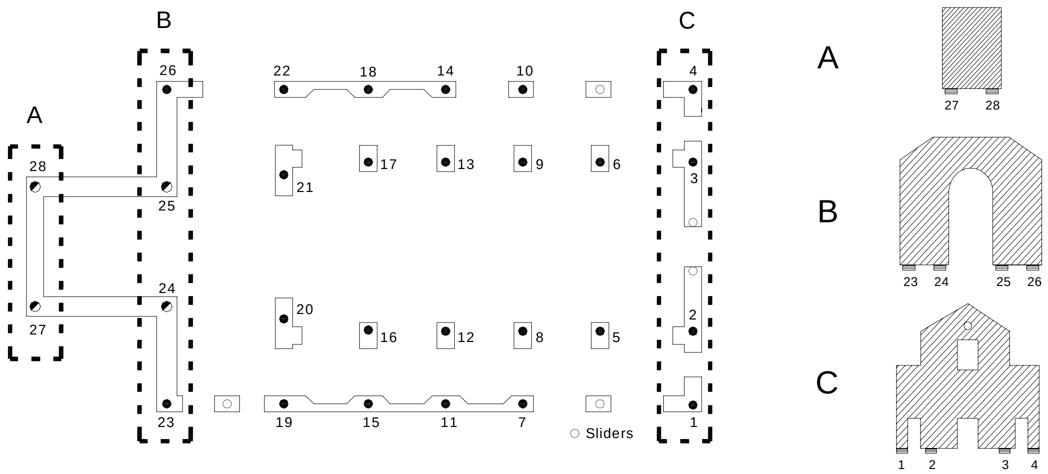
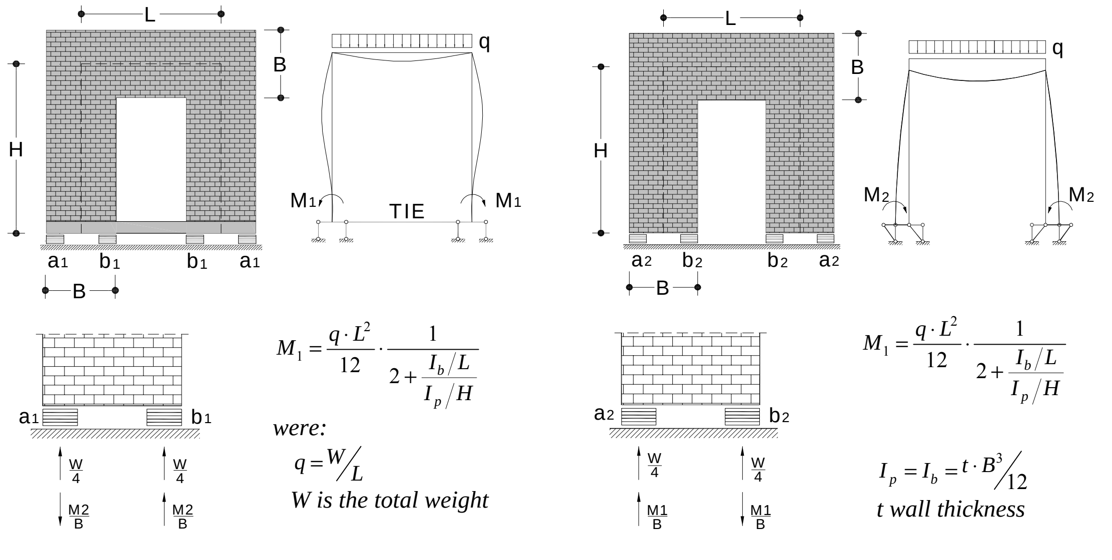


Figure 5 : Macroelements in SI church with major scatters between vertical stresses.

In fact, the rigid floor at the isolation level acts as a rigid tie applied at the base of the macro-element opening. This effect can be clearly explained through the simplified models of Fig. 6 a and b, representing a single portal frame with two bearings under each pier, (a) with, (b) without rigid slab at the base: it can be observed that the rigid slab does not allow the relative movement of the pier bases, giving rise to a bending moment reaction which is transferred to the bearing couple, located under each pier, as axial load. In absence of rigid slab, the reaction at the pier base, and consequently the axial loads transferred to the bearings, assume different values.



(a) (b)

Figure 6 : Simplified models: portal (a) with rigid tie, (b) without rigid tie..

In order to assess the influence of the deformability of the upper-structure according to the presence of a rigid slab at the base or not, a parametric analysis has been carried out on the simplified models. Different values of the opening ratio (i.e. the ratio of the opening width (L-B) to the piers width (2B)) have been considered with a step-wise variability of 0.5, 0.75, 1 and 1.25. In Fig. 7 the ratio between the values of axial load in the two models (with/without rigid tie), acting on the couple of isolators a and b, located under a pier, is provided as a function of the portal opening ratio. It can be observed that the variation can reach values greater than 30%.

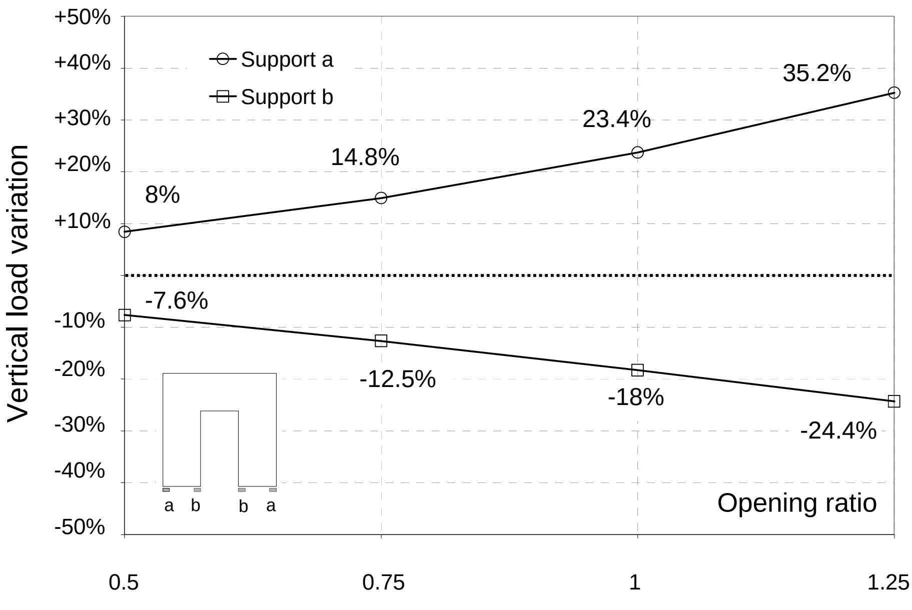


Figure 7 : Effect of the deformation in the upper-structure.

The macroelement B of the SI church, represented in Fig. 5, is quite similar to the simple portal model employed in the parametric analysis; the macroelement B is characterized by an opening ratio approximately equal to 0.5, and the ratio between vertical stresses of the two models (with/without rigid floor) are equal to 5.4%, 7%, 8% and 10%, respectively on the isolators 23, 24, 25 and 26. The diagram of Fig. 7, in the case of the simple portal model with opening ratio equal to 0.5, provides values of the stress ratio equal to 7.6% and 8%, (respectively for the support b and a) which are quite close to the values obtained for the macroelement B.

It seems that the above considerations have important design implications; in fact, when the artistic feature of the church does not allow the construction of the r.c. floor at the isolation plan, local stiffening interventions and simple ties can be placed at the base of the macroelements; however, the effect of the deformability of the macroelement-tie system should be taken into account when the axial load acting on isolators, due to the only vertical load condition, is computed.

6 CONCLUSIONS

In this paper, some design aspects on the base isolation system have been evaluated. A dimensioning procedure and a list of checks on the devices have been tested on two study cases, namely the church of St. Giovanni a Mare and the church of St. Ippolisto. Modal and time history analyses of the base isolated churches have been carried out. They have allowed to define vibration modes, periods and participating masses, shear distribution among the isolators and the church macro-elements, the maximum and minimum axial stresses in the isolators.

In particular the attention has been focused to assess the differences between two models, with and without rigid diaphragm connecting the isolators. The presence/absence of the rigid floor does not imply variations in terms of accelerations, displacements and forces in the upper part of the construction, but only different values of the normal stress on the isolators for vertical load, due do the different deformability of the upper-structure. Simplified models for describing this effect have been presented and a parametric analysis has shown that, for church masonry macro-elements characterised by very large opening ratios, the vertical stress on isolators significantly varies considering the presence/absence of rigid floor at isolation level.

As a design implication of the above observations, it can be stated that, when the artistic feature of the church does not allow the construction of the r.c. floor at the isolation plan, local stiffening interventions and simple ties can be placed at the base of the church macroelements, provided that the effect of the deformability of the macroelement-tie system is taken into account.

ACKNOWLEDGEMENTS

This research has been supported by RELUIS “Rete di laboratori Universitari Ingegneria Sismica” in the context of the activities of “Linea 7”.

REFERENCES

De Luca A., Mele E. 1996. The seismic isolation in the retrofit of historic buildings, *Proc. of USA-Italy Seminar on the Seismic Restoration of Historic Buildings,* Los Angeles, July 22-25.

Doglioni F., Moretti A., Petrini V. 1994. Le chiese e il terremoto. *Ed. LINT*, Trieste.

Elsesser E., Naaseh S., Walters M., Sattary V., Whittaker A.S. 1991. Repair of five historic buildings damaged by the Loma Prieta earthquake. The Seismic Retrofit of Historic Buildings Conference Workbook, San Francisco, California, 4-1 - 4-40.

Gulkan P. 1993. Report on the International Colloquium on Seismic Protection of Historic Buildings and Monuments, held in Quito, Ecuador, 31May-3June, *Getty Conservation Inst.*, Cal.

Lagomarsino, S., and Podestà, S., 2004. Seismic Vulnerability of Ancient Churches: I. Damage Assessment and Emergency Planning, *Earthquake Spectra* 20 (2), 395–412.

Naeim, F. & Kelly, J. M. 1999. Design of seismic isolated structures - From Theory to Practice. J. *Wiley & Sons*, New York.

OPCM 3431/05. Norme tecniche per il progetto la valutazione e l’adeguamento sismico degli edifici. *Presidenza del Consiglio dei Ministri*, Italy.

Proietti G., 1994, Dopo la Polvere, *Ministero per i Beni Culturali e Ambientali (5 Tomi), Istituto Poligrafico e Zecca dello Stato*, Roma.