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INNOVATIVE RESTORATION OF THE APAGNI ROMANESQUE CHURCH, DAMAGED BY THE 1997 MARCHE-UMBRIA EARTHQUAKE

M. INDIRLI[1], A. VISKOVIC[2], M. MUCCIARELLA[3], C. FELEZ[4]

SUMMARY

Conservation criteria of Masonry CUltural HEritage Structures (MCUHESs) are often not compatible with correct antiseismic requirements. In fact, the use of conventional methods leads, in most cases, to retrofitting interventions excessively invasive or ineffective. This conflict has been unfortunately demonstrated by the relevant damage found in a large amount of MCUHESs, restored after earthquakes but again seriously struck by subsequent seismic events. Due to those simple statements, a different approach to the question has been pointed out, addressing to the use of modern antiseismic techniques; they can reduce the dynamic actions transmitted by the earthquake, rather than improve the structural resistance. After a detailed diagnostics and monitoring campaign, the Romanesque church of San Giovanni Battista at Apagni (Sellano, Perugia) has been selected as a pilot application of Seismic Isolation (SI) by means of a specific subfoundation system, in order to respect as better as possible the building original features. This paper shows the main steps of the preliminary work and the design proposal.

INTRODUCTION

MCUHESs protection of many ancient (and frequently precious) buildings coming from the past is not an easy question for seismic-prone countries like Italy. In fact, MCUHESs are very vulnerable under seismic excitations, because even moderate events can cause collapse or heavy damage. Several existing still standing MCUHESs, even not yet severely damaged, could be at least weakened by previous earthquakes. In addition, their resistance has been lowered by other factors, such as chemical attacks to masonry due to air pollution and traffic-induced vibrations. Thus, there is the urgent need for many of them to be seismically rehabilitated or at least, "improved", in order to make them capable to withstand future earthquakes without collapsing or becoming affected by severe damage. Unfortunately, the MCUHESs rehabilitation problems are much more difficult to solve than those related to modern r. c. or steel structures, due to specific conservation criteria (integrity, compatibility, reversibility and durability) not easily reconcilable with seismic requirements. An additional problem for existing MCUHESs is that their characteristics (material properties, construction aspects, state of integrity) are frequently not very well

[1] ENEA – "E. Clementel" Research Center, Bologna, Italy, maurizio.indirli@bologna.enea.it
[2] "G. D'Annunzio University" – Chieti-Pescara, Italy, alberto.viskovic@tin.it
[3] Architect – Rome, Italy, iltrilite.2000@tin.it
[4] Studio Croci-SPC – Rome, Italy, mail@spc-enginnering.com

known, so that, for instance, a localized intervention (e.g. strengthening) might cause even more severe damage to other parts of the structure in the next earthquake. This and other factors make each MCUHES different from the other ones; therefore it is necessary to undertake the rehabilitation design in a specific way, use of standardized procedures being not possible. In order to avoid the above-mentioned conflict between conservation criteria and seismic requirements, an innovative approach consists in the reduction of the dynamic actions transmitted by the earthquake to the structure by using SI, rather then strengthening. SI, greatly developed in the last 25 years and now a fully matured technology, is based on the idea of reducing the seismic input energy by changing the structure's dynamic characteristics, i.e. increasing its natural period, in order to make it farther from the period of the main harmonic components of the seismic actions (Fig. 1). This change is usually achieved through the use of special devices (isolators) with very low horizontal stiffness and appropriate damping, that "decouple" the structure from the motion induced by the earthquake. SI allows the mitigation of the effects of the earthquake not only on the structure itself, but its contents as well. SI is named "Base Isolation" (BI) when the isolators are interposed between the building and its foundations [1].

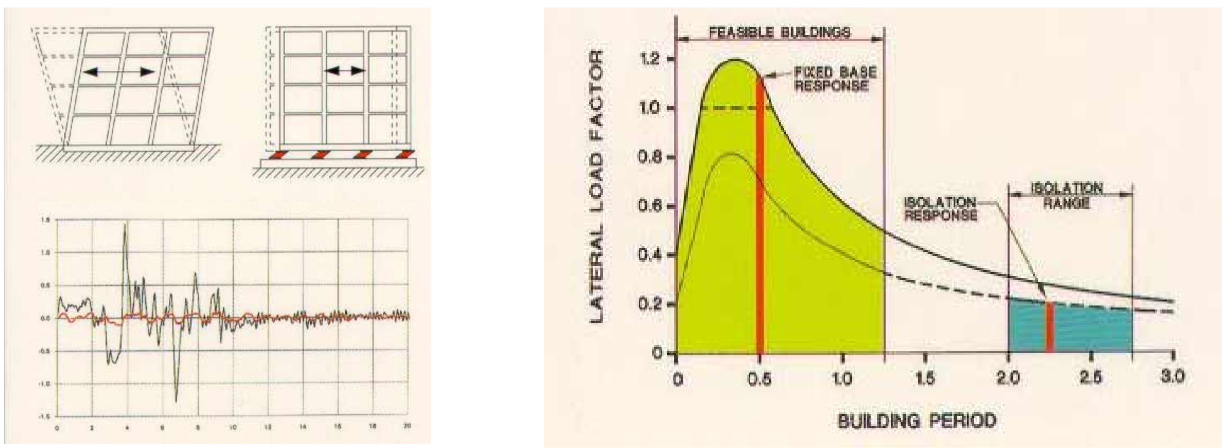


Fig. 1. Non-SI and SI behavior scheme, with an evident seismic response reduction (left); natural period shifting far from the most energetic seismic excitations (right)

Amongst hundreds of applications, BI was first used in a MCUHES in 1987/88 (retrofitting of the Salt Lake City and County Building, USA). Other examples are present in USA (Oakland, Los Angeles and San Francisco City Hall; Museum of Asian Art in San Francisco) and New Zealand. Some applications of modern antiseismic techniques to MCUHESs are now underway also in Italy, including BI [2].

After detailed diagnostics and monitoring campaigns, the Romanesque church of San Giovanni Battista at Apagni (Sellano, Perugia) has been selected as a BI pilot application by means of a specific subfoundation system, in order to respect as better as possible the building original features. The church was damaged by the 1979 Valnerina earthquake, after which it had been restored with conventional methods. This restoration, although correctly performed, was clearly insufficient as to protect the church against the subsequent 1997-1978 Marche-Umbria. The church, again heavily damaged and declared unsafe, was immediately subjected to a prompt intervention and rehabilitation works.

DESCRIPTION OF THE SAN GIOVANNI BATTISTA CHURCH AT APAGNI

The Romanesque Apagni church of San Giovanni Battista [3-4] is a frescoed single-room ecclesiastical structure, with a shed roof and a sail bell-tower ("*vela campanaria*"), typical of the stone churches found in the Marche-Umbria Apennines. The church, placed in the municipality of Sellano (Perugia), in the hamlet of Apagni, is part of the cemetery. The graveyard chapel is adjacent to the church. There is a lack of relevant historical data, with regards to the construction phases. A probable period of construction (between 1300 and 1600) is that in which the frescoes were realized, of which fragments remain on the interior; in fact, art historians have discovered in them the hand of the Angelucci, an artists family active in Marche and Umbria in the XVI century. It is possible to suppose ancient works due to the repairs necessitated by damage caused by seismic activity, pointed out by the presence of angle stiffeners.

The church lays on a gentle slope, so the walls have different height. The dimensions are the following: length 13.50 m (externally) and 12.00 (internally); width 8.30 m (externally) and 7.00 m (internally); the external height varies from 5.6 m at the gutter line to 7.10 m at the ridge line, while the maximum height of 8.00 m is at the East corner, on the back side. Two minor bodies lean against the main church building: a small chapel contemporary to the church construction on the right hand side (3.15 x 1.80 m), which is the basement of the sail bell-tower wall, made of squared stones, with two arched vaults (of differing diameters, 95 and 80 cm, in brickwork) terminating in a pitched roof; the more recent sacristy (in juxtaposition with the posterior side and in correspondence with the choir), which was most likely destroyed by the 1979 earthquake and subsequently reconstructed with light concrete. The main façade, with its shed roof, presents, along its center line, a series of voids (a portal with a vaulted arch surmounted by a round window). The roof (with a clay roof-tiles mantle) is built of triangular wood trusses, with secondary framing in wood beams and flat clay tile infill. Internally, the chancel zone (presbytery) is raised, with a flat termination with a triumphal arch on the end wall. The masonry walls are made by a double wall structure (total wall thickness about 70-80 cm) with a rubble fill core ("muratura a sacco", a masonry technique in which two external faces of stone are held together by a nucleus of lime based mortar and broken bits of stone). The walls (anciently completely frescoed) are covered by a cement rendering coat, on which traces of frescos can still be seen, especially in the chancel zone. The limestone paving is the cover for the charnel-house (ossuaries), apparently vaulted, accessible by seven regularly spread trap-doors, four at one side and three at the other. The material used is local white-colored limestone, with low-medium mechanical properties.

SEISMIC HISTORY AND HAZARD, EARTHQUAKE DAMAGE AND RESTORATIONS

The principal historical seismic events which interested Valnerina and the town of Sellano are shown by Table 1 [5]. The Marche-Umbria earthquake sequence started on September 26th, 1997 and took place in a complex deforming zone, along a normal fault system in the Central Apennines. The seismic sequence left significant ground effects, which were mainly concentrated in the Colfiorito intermountain basin. The crustal events generated extensive ground motion and caused great damage in several urban areas. The extent of macroseismic data and the abundance of recorded ground motions permits a good knowledge of the source and structural parameters to better understand the nature of the ground shaking and the resulting damage patterns. Three main shocks (time 2:33, M_L 5.5 and VIII MCS; time 11:40, M_L 5.8 and VIII-IX; time 11:46, M_L 4.7 and VII) hit with epicenter near Cesi and Collecurti (lat. 43.0; long. 12.9), towns located on the border between Marche and Umbria Regions.

These shocks were also responsible of the vaults collapse in the Assisi St. Francis Upper Basilica. The seismic crisis lasts several months; during subsequent events, other towns were struck, causing heavy damage in many MCUHESs: Nocera Umbra (October 3rd, time 10:55, M_D 4.8 and VII MCS; October 4th, time 18:13, M_D 4.3 and VI); Casenove and Forcatura (October 7th, time 1:24, M_D 4.9 and VII-VIII; time 7:09, M_D 4.1 and V-VI); Sellano and Preci (October 12th, time 13:08, M_D 4.5 and VI-VII; time 17:23, M_D 4.9 and VII-VIII).

Before the seismic sequence, probabilistic and deterministic maps were available. The first indicates, for the Marche-Umbria region, Peak Ground Accelerations (PGA) not exceeding 0.4g, for 475 years return period (Fig. 2), and 0.24g, for 100 years return period [6]. A first-order deterministic seismic zoning of Italy lead to theoretical peak values of Design Ground Acceleration (DGA), as shown in Fig. 3 [7].

Place	date			Lat.	Long.	I (MCS) epicenter	M_m
	day	month	year				
Etruria	-	-	217 B. C.	43 250	11 250	10.0	6.6
Sabina	-	-	174 B. C.	42 250	12 670	10.0	6.6
Norcia	-	-	99 B. C.	42 800	13 100	9.0	5.4
Rieti	-	-	76 B. C.	42 400	12 870	10.0	6.6
Spoleto Roma	29	April	801	41 900	12 480	7.5	5.1
Spoleto	-	-	1246	42 732	12 736	7.5	5.1
Spoleto	-	-	1277	42 732	12 736	8.0	5.4
Reatino	1	December	1298	42 550	12 830	9.5	5.8
Norcia	1	December	1328	42 856	13 018	10.0	6.6
Viterbese-Umbria	9	September	1349	42 620	12 120	8.5	5.8
Foligno	2	February	1477	42 955	12 704	7.5	5.1
Spoleto	-	-	1667	42 732	12 736	7.0	4.8
Norcia	18	October	1702	42 833	13 083	7.0	4.8
Spello	14	November	1702	42 917	12 667	7.0	4.8
Rieti Appennines	14	January	1703	42 680	13 120	11.0	7.1
Spoleto	29	June	1703	42 750	12 750	7.0	4.8
Spoleto	20	May	1704	42 750	12 750	7.0	4.8
Narni	-	-	1714	42 517	12 521	7.5	5.1
Cascia	4	October	1716	42 750	13 000	7.0	4.8
Norcia	12	May	1730	42 752	13 117	9.0	5.8
Spoleto	-	March	1745	42 732	12 736	8.0	5.1
S. Gemini	11	June	1751	42 594	12 593	7.0	4.8
Gualdo Tadino	27	July	1751	43 222	12 730	10.0	6.6
San Gemini	26	May	1753	42 617	12 550	7.0	4.8
Umbria	25	December	1766	42 750	12 917	7.0	4.8
Spoletino	5	June	1767	42 820	12 750	7.5	5.1
Piediluco	9	October	1785	42 564	12 777	8.0	5.4
Norcia	3	September	1815	42 756	13 054	7.5	5.1
Rieti	22	March	1821	42 417	12 833	7.0	4.8
Foligno	13	January	1832	42 967	12 659	8.5	5.8
Valnerina	14	February	1838	42 875	12 886	8.0	5.4
Spoleto	22	September	1853	42 683	12 667	7.0	4.8
Norcia	22	August	1859	42 825	13 097	8.5	5.8
Visso	15	August	1884	42 933	13 083	7.0	4.8
Umbria-Marche Appenn.	18	December	1897	43 500	12 380	7.5	4.8
Rieti	27	June	1898	42 415	12 905	8.0	5.1
Visso	25	August	1898	42 910	12 973	7.0	4.8
Valnerina	2	November	1903	42 794	13 074	6.5	4.6
Gualdo Tadino	31	July	1914	43 200	12 800	7.0	4.8
Assisi	26	March	1915	43 070	12 616	7.0	4.6
Sellano	10	March	1941	42 888	12 926	5.5	4.0
Deruta	3	November	1941	43 000	12 433	7.0	4.8
Rieti	2	February	1963	42 383	2 950	7.0	4.8
Preci	2	August	1964	42 835	13 036	7.0	4.6
Trasimeno	11	August	1969	43 036	12 226	7.0	4.8
Sellano	7	September	1970	42 867	12 950	6.0	4.3
Valnerina	19	September	1979	42 720	13 070	8.5	5.8

Table 1. Principal seismic events interesting the Sellano area.

The 1979 Valnerina earthquake damage and intervention

The 1979 earthquake (epicenter between Norcia and Cascia) caused serious damage to several MCUHESs, including the Apagni church. Sellano Town Hall and the Umbria Fine Arts Office performed and realized several projects to restore damaged structures and increase their earthquake resistance.

Sellano was chosen as one of the pilot area in which typical interventions were done, such as the insertion of concrete walls and edges, without reinforcing the existing masonry walls, very vulnerable to seismic actions. Thus, the Apagni church had been restored with conventional methods: the intervention interested especially the walls and the cover. The walls were reinforced by means of cement injections from the external side only. The cover was rebuilt using materials very similar to the original ones (i.e. wooden roof trusses and squared beams) and roof-tiles. A concrete slab was also cast on the roof-tiles and the waterproofing. Typical roman tiles were finally placed. Concrete string-courses were realized on each wall, but at different levels, so the connection between them could not be effective. This restoration, although correctly performed and formally in line with the existing codes at the time of the intervention, was clearly insufficient as to protect the church and turned out to be counterproductive when the building was struck again by the 1997 earthquake.

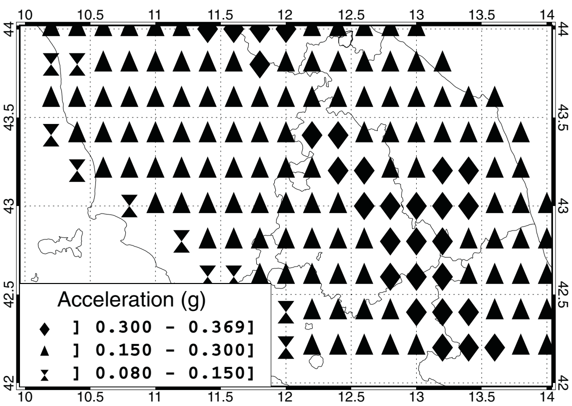


Fig. 2. Probabilistic estimation of PGA

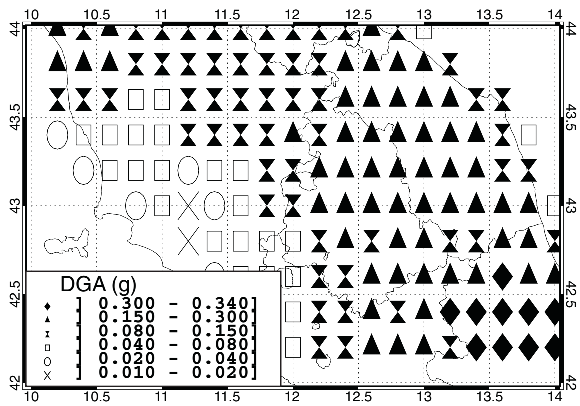


Fig. 3. Deterministic DGA

The 1997 Marche-Umbria earthquake damage and intervention

The new seismic event (classified as moderate) damaged the Apagni church again (Figs. 4-5) and the church was declared unsafe and closed. The cracks present on the structure pointed out the effects of the interventions realized in the 80's. Under seismic excitation, the architectonic organism (in addition to several typical injuries due to its intrinsic structural vulnerability, as the falling down of the bell-sail, rotation of the front, additional parts separation, widespread presence of cracks near the openings), gave an unexpected answer, showing abnormal damage. The large cracks allowed to verify the walls width in which the injections were effective and the injection points. The reading of fissures and lesions was interesting from a didactic point of view. The typical seismic excitation shear-force response of the entire single-room building should have been the opening and overturning towards the exterior. However, the insertion of the concrete string-courses and the cement injections (done in the previous restoration) conferred different stiffness to the masonry wall structure along its verticality; consequently, the entire building was roughly divided by the horizontal shear-force into three parts, which moved reciprocally amongst themselves with different behavior: the lower part largely resisted, conserving almost entirely intact its “a sacco” masonry; the upper part (concrete string-courses, triangular trusses and roof), separated itself completely from the underlying masonry and rotating autonomously; therefore, the central part experienced relevant torsion stresses, due to the differential movements between the inferior and superior portions. The “a sacco” masonry suffered a general loss of cohesiveness and extensive detachments; a typical damage in blocks, due to the walls non-uniform resistance (subsequent to the above-mentioned injections), was evident; the masonry walls fractured exactly along the reticular lines along which the injections were executed (i.e. corresponding to the still in-situ pvc-nozzle positions).

The façade suffered a visible detachment from the longitudinal walls (slight planar rotation/translation and plastic hinge formation at the height of the of the lightly buttressed section). Through-passing cracks (vertical and oblique, singular and multiple) were present, especially in the central strip including the voids; many punctual collapses occurred, including those of the portal and the arch keystones; the four keystones of the central circular window underwent downward shift and planar rotation; the two corners, immediately below the concrete string-course, suffered twisting movements along two axes and evident rotation (causing several detachments), due to the interaction between the orthogonal masonry panels and the hammering of the stiffened roof (added during the latter restoration).

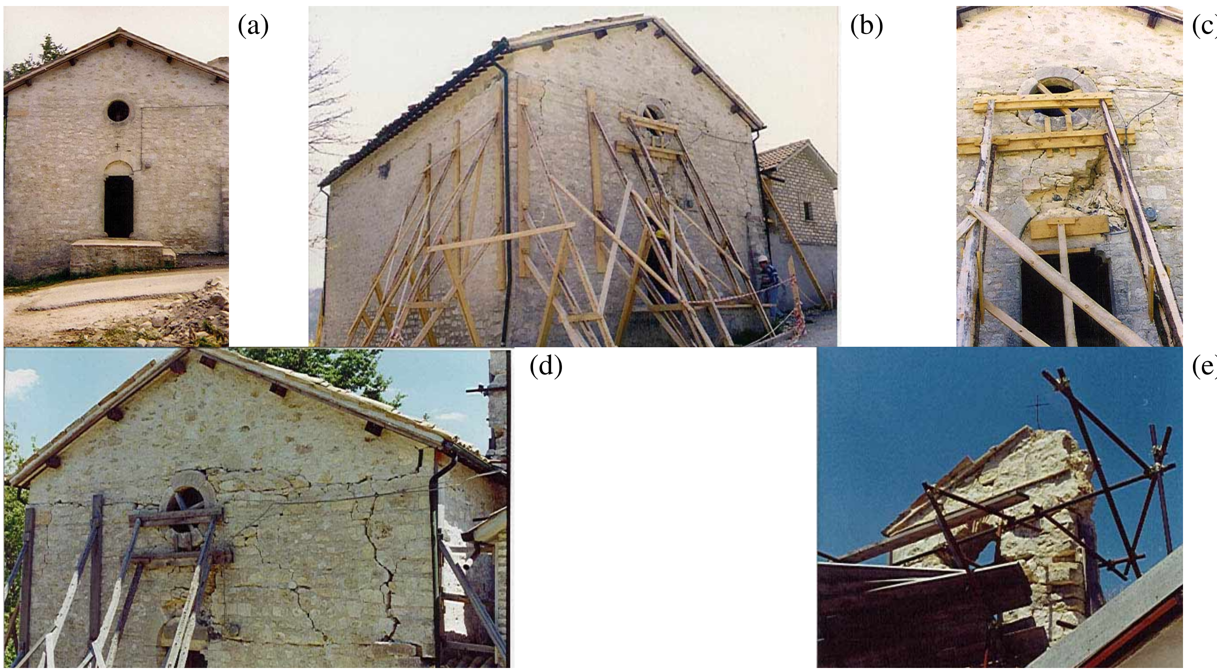
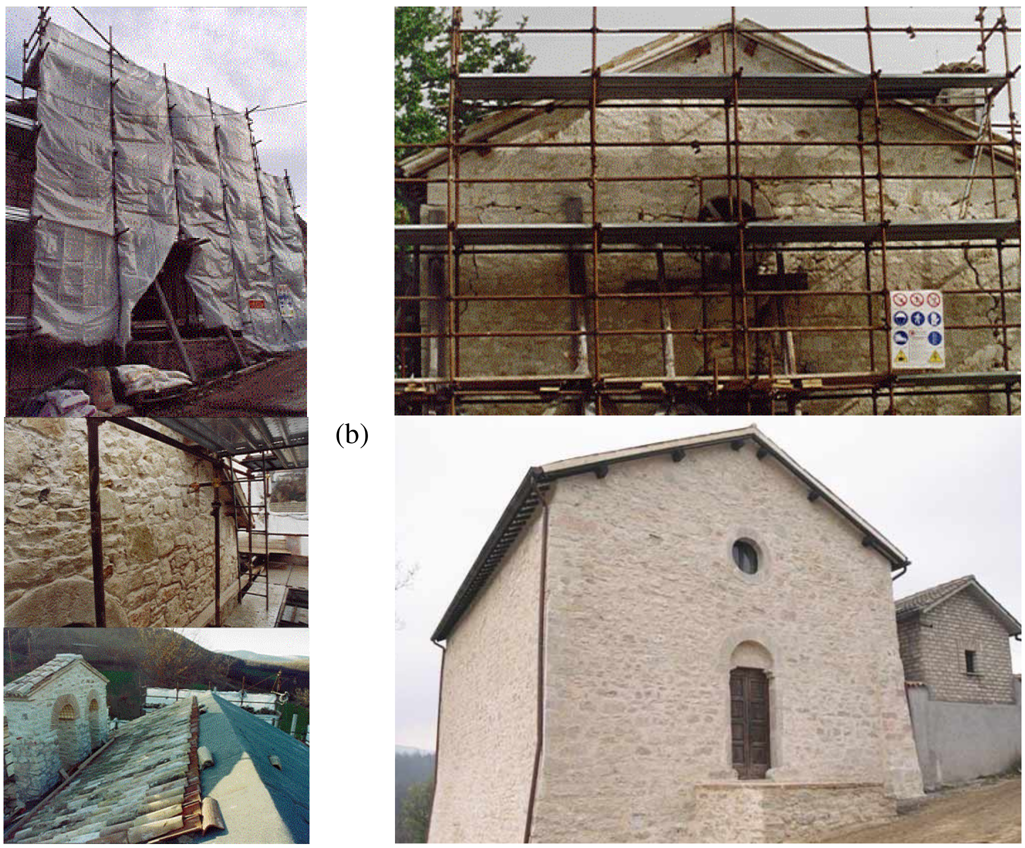


Fig. 4. The Apagni church: (a) before the 1997 earthquake; (b) (c) (d) (e) 1997 seismic damage

Fig. 5. Internal damages to the church and frescoes

As mentioned before, the lateral above-wall is less tall and close to two minor cemetery chapels, one connected and the second (funereal chapel) not-connected; on the other side, the below-wall is higher and completely free. Therefore, the suffered damage was different in the two sides. The below-wall moved and divided itself into three parts and showed perfectly horizontal cracks. Due to the loss of the upper connection (concrete string-course detachment), it remained connected to the remaining three sides, flexing freely and showing positive out-of-plumbs in the corners and negative in the middle; the failure-summary regarded inclined through cracks in the depressed central portion, oblique cracking due to the façade and end-wall rotation. The out-of-plumb highest values occurred in the corner connected to the façade (about 10 cm in the orthogonal direction and 4 cm in the parallel one). The above-wall presented minor cracks; the loss of the roof-wall connection was shown by an horizontal cut and a misalignment of about 3 cm. Oblique cracks presence attested the rotation of the two orthogonal wall panels. Detachments were suffered by the annexed chapel (about 1-2 cm) and the non-connected funereal chapel (about 4-5 cm); the latter experienced also strong cracking, due to the weak-bonding between the wall concrete blocks. Both the structures suffered roof damage, due to the sail bell-tower collapse (some of its stone elements fell down during the earthquake).



(a)

(b)

(c)

Fig. 6. (a) Prompt safety intervention, (b) rehabilitation works and (c) church after the conventional intervention

The very notable damage to the sail bell-tower was caused by its specific vulnerability. The free portion oscillated largely (due to its own acceleration and bell-swinging), and the last small column and the right-hand arch collapsed, falling on the roofs of the two small chapels, as described above. The two remaining-in-place columns were completely cut along an horizontal plane, separating themselves into three parts; each of these parts underwent torsion rotation, causing the expulsion of some stone elements and misalignments in the longitudinal (5 cm) and orthogonal (3 cm) directions. The remaining-in-place arch key-bricks slipped downwards (about 6-7 cm). The presence of reinforcements (threaded bars insertion into the columns) avoided the overall collapse of the structure.

The (improperly named) apse, more precisely a body joined to the sacristy, suffered global detachment and translation of the upper part, along a perfectly horizontal plane (misalignment of about 3 cm). Due to its reconstruction in concrete blocks, the shear-fracture mechanism induced horizontal cracking (in correspondence with the joints) and below-corner expulsion (translating along two axes of about 5 cm). The more ancient basement, better tied to the existing masonry of the main room, remained undamaged. The translation movement involved also the external masonry side of the end wall (misalignment of about 3 cm of the part included between the annexed building roof and the string-course level).
The triumphal arch underwent a geometric deformation, with a shear fracture of the two shoulders (height of about 60 cm above the floor level), collapse of the spring point, punctual failure of the midpoints and keystone lowering.
The cracking summary of the internal walls was in line with that of the exterior, because the most of the cracks were passing through. Also in the façade and below wall interior, the subdivision into three horizontal parts was clearly legible. In addition, the oblique cracks (due to the façade rotation/translation, sacristy translation and free wall inflexion) were still found in the two longitudinal walls.

The 1998 prompt safety intervention and conventional rehabilitation

Above all, the project foresaw the scaffoldings construction around the church (Fig. 6a), in order to avoid further collapse of the crumbling masonry and assure the workers safety during the restoration. A metallic reticular structure (tubes and joints) was realized, offering the advantage of a swift execution.
The conventional rehabilitation (financial resources provided by public funds) was conceived in order to give back the wall structure its original stiffness and placing resistant systems capable of improving the building dynamic response to seismic shear-force actions (Fig. 6b). The designers decided to intervene according to the existing set of rules and respecting as best as possible the original church structure and components, in order to obtain a functional synergy with the existing elements and guarantee the reversibility. The intervention handled the critical situations assessed and outlined the following patterns: repairing damages and strengthen the weak points of the masonry structure; joining one each other roof, concrete string-course and walls; eliminating the mutual actions due to the additional parts.
The damaged masonry had to be repaired through "*cuci-scuci*" ("stitching and unstitching") method, in order to reorganize the structural continuity, realizing effective connections between new and old wall parts, utilizing stone elements rescued from the ruins and compatible mortar. In addition, a ring of metal tie-bars on each of the four walls was introduced, at the critical height between the portal and the round window, where the greatest damage occurred; the anchorage to the walls was realized in concealed stiffened steel plates and the four corners were reinforced with steel-stitching. The stone elements of the main portal and round window above had to be replaced in their former exact position. The collapsed triumphal keys of the arch and sail bell-tower had to be reused (if possible) or replaced with compatible materials. Reinforced perforations in the sail bell-tower base columns had to be carried out, in order to anchor them to the bell-tower, in addition to the tie-bar insertion in the arches spring point.
The connection between the roof string-course and the upper portion of the masonry walls had to be realized by a steel bars anchorage. The sacristy was reconstructed with self-supporting antiseismic blocks, in order to guarantee a better dynamic response. The new construction was isolated from the main hall by an antiseismic joint, realized in-situ with a compressive material. The roof had to be rebuilt in a load-bearing wood structure (strengthened by a reinforcing slab) and connected to the new masonry walls by reinforced concrete string-courses. Finally, all the mortar joints were deeply raked and refilled with a mortar studied specifically for that purpose, together with the Fine Arts Office, in order to assure a chromatic continuity with the environment. This intervention contributes to the enhancement of the mechanical characteristics of the masonry. Thanks to funds availability, it was possible to include in the restoration also finishing works (internal stucco-work and external walls cleaning) previously not planned, in order to give the church back to its community in a well-usable condition. The external walls cleaning was performed by low-pressure jets of water and successive application of a protective chemical product; residuals were removed by a final wash. The frescoes were in a worrisome state of detachment; thus, they

were subjected to “first aid” works, by using tissue-paper and glue. They were consolidated by injections underneath the painted and/or supporting surface; the tissue-paper was then removed and the residue cleaned. Two highly damaged frescoes, found in a out-of-plumb area, were removed and re-glued after being fixed to a backing panel. Fig. 3c shows the church after the intervention.

DYNAMIC CHARACTERIZATION AND SEISMIC MONITORING

The dynamic experimental analysis has been organized in three phases.
A first dynamic characterization after the 1997 earthquake allowed to have a first glance at the dynamic properties of the structure and define the optimal measurement points for the monitoring network. Eight seismometers Kinemetrix SS1, an HP3566A signal conditioner and a laptop composed the experimental set-up; the signals recorded by the eight seismometers, used in synchronized way, were collected by the acquisition system and analyzed in real time by HP software; two kind of vibration source were considered: ambient vibrations and an impulse produced by an impact of a mass dropped on the ground near the church. Transducers were temporarily installed in four different configurations. Several time-histories lasting 64 s were recorded for each configuration. This was done to show repeatability of the vibrational characteristics and to get average values of the characteristics. The recorded data were studied in the frequency domain by means of cross-spectral analysis. The motion in terms of modal shapes was examined by means of the PSD amplitudes. Peaks at 5.5 and 6.0 Hz were present, probably related to the modal shapes of the structure. Because of the presence of the scaffoldings, the dynamic behaviour of the church was very complex. This results gave only a first idea of the dynamic characteristics.

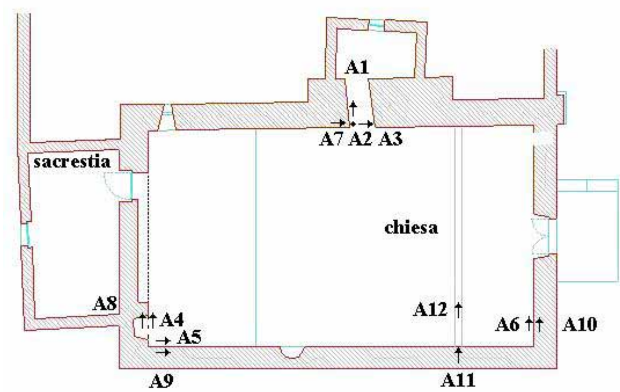


Fig. 7. Accelerometric network; A1, A2, A3: K2 internal tern; A4, A5, A6: on the wall base; A7, A8, A9, A10, A11: on the wall top; A12: on the roof truss

Seismic monitoring has been completed on the conventionally restored church; during March 2002, an accelerometric network (acquisition system Kinemetrics K2, with 3 internal channels and 9 external accelerometers Kinemetrics FBA11) has been installed; the K2 also contains a 20 Mb PCMCIA card, a battery and a tern of accelerometric sensors FBA; the location of the sensors is reported by Fig. 7. From March to September 2002 about 90 events have been recorded, then analyzed and classified depending on the energy content, evaluated through the Arias scalar intensity at the basement of the structure, assuming as reference the acceleration values recorded by the K2 internal tern (A1, A2 and A3).
All the data have been analyzed in order to verify the structural behaviour, found constant at varying of the input energy; this result demonstrates a linear trend in the obtained energy range and the effectiveness of the rehabilitation works. The spectral analysis provided the following results: in the transversal direction, resonance peaks are evident at 5.8 and 7.0 Hz (the latter with ordinates higher than the further in the top sensors); in the longitudinal direction, resonance peaks are evident at 6.0 and 7.0 Hz; peaks corresponding

to 3.15 e 3.4 Hz frequencies are due to the soil, which doesn't present amplification. The cross-spectra analysis showed that in the transversal (A4-A8 e A6-A10) and in the longitudinal walls (A3-A7 e A5-A9) resonance peaks are evident at 6.0 e 7.0 Hz. The same values of the resonance frequencies has been noted in the cross-spectra of the sensors located on the top of the transversal walls (A8-A10 e A7-A9); in the first one, the first value appears shifted at 5.8 Hz. The modal shape associated to the 6.0 Hz frequency is characterized by a rotation around an external point of the church plant (façade side). The contrary happens for the second modal shape.
Another experimental canpaign of dynamic characterization is foreseen after the completion of the BI application, in order to verify the changed structural behaviour of the church.

(a) (b)

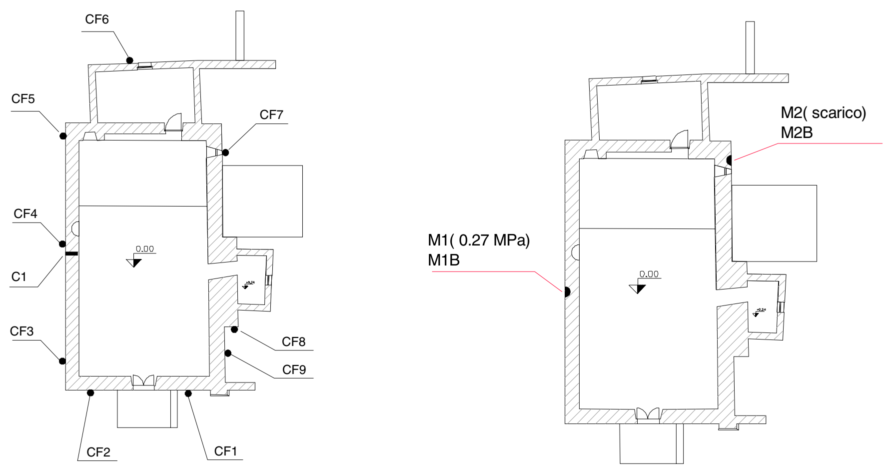


Fig. 8. Location of the horizontal and sub-vertical drillings (a) and flat jacks tests (b)

EXPERIMENTAL CAMPAIGN OF DIAGNOSTICS INVESTIGATION

The work has been carried out on masonry walls and foundations by ENEL.HYDRO, performing the following tests (Fig. 8-10): continuous drillings on the rising walls (label Cx), detailed stratigraphy, TV color probe survey; continuous drillings on the foundation walls (label CFx), detailed stratigraphy, TV color probe survey; tests with single flat jack, in order to determine the stress conditions (label Mx); tests with double flat jacks, in order to determine the deformation properties (label MxB).

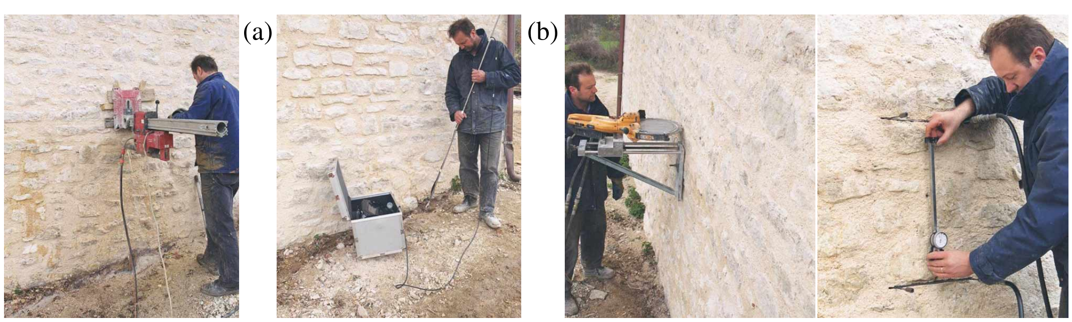
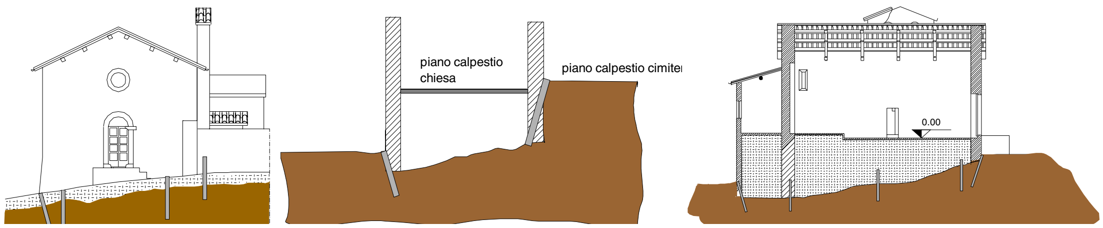


Fig. 9. Tests execution: (a) drillings, (b) survey with TV color probe, (c) flat jacks

The masonry structure of the raising walls is made by blocks of medium-large dimensions in the two external faces and a rubble fill core ("muratura a sacco") composed by smaller and less regular stones with discontinuities and cavities; the mortar is well distributed in the vertical faces, less accurate in the internal foundation layers. The foundations are shallow (20-30 cm) and immediately under the ground surface, except at the West (250 cm) and South-East sides, where the laying line seems to be deeper, due to the presence of filling soil moved throughout the time in order to realize respectively the cemetery level and a small country road. Along all the church perimeter, the soil, investigated until -3 m, is constituted by clay. The stress conditions, compatible with the structural dimensions, are 0.27 MPa on the East church wall, while the West side is unloaded; those results are certainly influenced by the different elevation levels. The masonry walls have medium-high deformation properties, little better in the East side with respect to the West side (30-40% variation).



(a) transversal (b) transversal (c) longitudinal

Fig. 10. Sections and profiles of the foundations

horizontal acceleration						
$0 \le T < T_B$	$S_e(T) = a_g \cdot S \cdot \{1 + T/T_B \cdot (\eta \cdot \beta_0 - 1)\}$					
$T_B \le T < T_C$	$S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0$					
$T_C \le T < T_D$	$S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \cdot (T_C/T)$					
$T_D \le T$	$S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \cdot (T_C T_D/T^2)$					
Soil category	S	T_B	T_C	T_D (*)	β_0	η
medium consistency	1.25	0.15	0.50	2.0	2.50	$\{10/(5+\xi)\}^{1/2} \ge 0.55$
B	*(*) in case of BI, T_D = 2.5*					ξ factor of equivalent viscous damping
vertical acceleration						
$0 \le T < T_B$	$S_{ve}(T) = 0.9 \cdot a_g \cdot S \cdot \{1 + T/T_B \cdot (\eta \cdot \beta_0 - 1)\}$					
$T_B \le T < T_C$	$S_{ve}(T) = 0.9 \cdot a_g \cdot S \cdot \eta \cdot \beta_0$					
$T_C \le T < T_D$	$S_{ve}(T) = 0.9 \cdot a_g \cdot S \cdot \eta \cdot \beta_0 \cdot (T_C/T)$					
$T_D \le T$	$S_{ve}(T) = 0.9 \cdot a_g \cdot S \cdot \eta \cdot \beta_0 \cdot (T_C T_D/T^2)$					
horizontal displacement						
$d_g = 0.025 \cdot S \cdot T_C \cdot T_D \cdot a_g$						
Soil category	S	T_B	T_C	T_D (*)	β_0	η
medium consistency	1.00	0.05	0.15	1.00	3.00	$\{10/(5+\xi)\}^{1/2} \ge 0.55$
B	*(*) in case of BI, T_D = 2.5*					ξ factor of equivalent viscous damping

Table 2. Italian code expressions for the elastic response spectra

INTERVENTION WITH BASE ISOLATION AND SUBFOUNDATIONS

Seismic action

The design of intervention, still in progress, makes use of the recently developed new Italian seismic code, together with the seismic reclassification of the Italian territory [8]. The municipality of Sellano is now classified in Zone 1, characterized by a Peak Ground Acceleration (PGA) value of a_g = 0.35 g. After selecting the soil stratigraphic profile for the Apagni church (medium consistency B), the Italian code gives the expressions of Table 2 in order to obtain the elastic response spectra.

Analysis methodology

Still in progress calculations are carried out on Fixed Base Structure (FBS) and Base Isolated Structure (BIS) through static, frequency and response spectrum analyses by using ABAQUS code. The two horizontal seismic components are considered acting simultaneously (100% for the x direction, 30% for the y direction and vice versa). The vertical component is always considered at 100%. In case of BIS, the response spectra have been reduced taking into account the factor of equivalent viscous damping of the isolation system (ξ_{esi} = 10%, while ξ = 5% in case of fixed base). A Finite Element Model (FEM) of the FBS has been developed (1672 nodes e 1026 elements, see Fig. 11), calibrated on the basis of the experimental data. The FEM is constructed by linear solid elements (principal walls, sail bell-tower, tympana), shells (floors, foundation walls, sacristy, chapel) and beams (ridge-beam, truss-beams and ties). Model total mass (6900 kN) and center of mass coordinates (x_g=6.42 m, y_g=4.74 m, z_g=4.01 m, x = longitudinal axe, y = transversal axe, z = vertical axe, Fig. 11) have been calculated. Foundation walls (about +5%) and charnel-house (about 80 kNs^2/m) masses were also considered. In the BIS model, the r. c. foundation string-course has been added and assumed for the isolators a spring behavior. Modal analyses permitted to individuate modal shapes, natural frequencies and participation factors, respectively for FBS and BIS (Fig. 12).

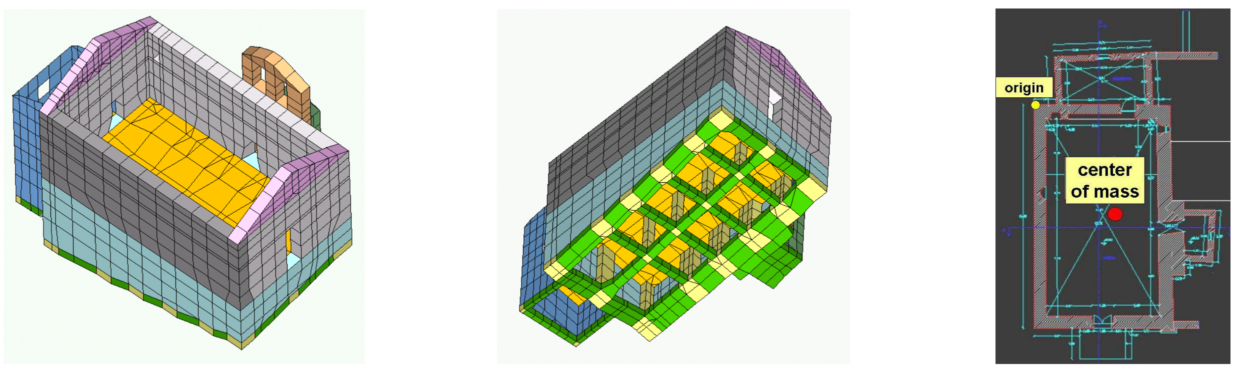


Fig. 11. FEM and center of mass position

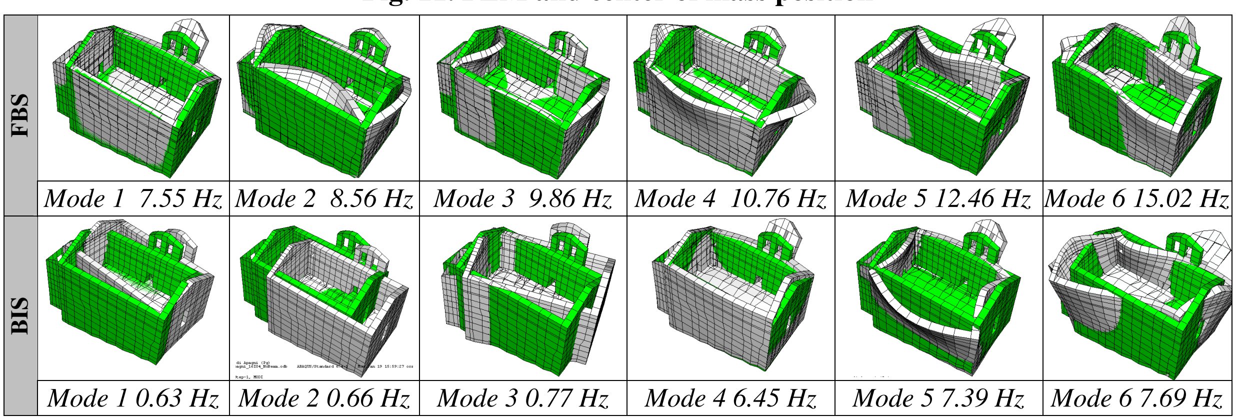


Fig. 12. Modal shapes for FBS and BIS models

Base Isolation (BI) system

The BI system adopted in the preliminary calculations (Fig. 13) is constituted by 8 identical High Damping Rubber Bearings (HDRB) and by 3 couples of Sliding Devices (SD) under sacristy, chapel and entrance portal. If about 1/5 of the structural mass is supported by SDs, the vertical load acting on each HDRB is 755.4 kN. Refining the design, different dimensions and stiffness will be selected for the HDRBs, depending on the position under the church, in order to minimize the distance between the structural center of mass and the BI center of stiffness (reduction of the torsional effects). Knowing the structure total mass M and assigning a first attempt value to the BIS isolation period (T_{is}=1.5 s), the total equivalent horizontal stiffness of the isolation system (K_e=1349.7 kN/m) is obtained from the expression $T_{is}=2\pi\ (M/K_{esi})^{1/2}$. Thus, the horizontal stiffness of the single isolator is K_{esi}=10797.4 kN/m. Taking from the design spectra a displacement of 0.20 m, diameter and height (i.e. area) of the bearing can be get from the expression $K_e=G_{din}\ A/t_e$, where: G_{din} shear dynamic equivalent modulus, A surface of the single elastomeric layer after deducting eventual holes, t_e total rubber layers thickness. The equivalent viscous damping is given by the expression $\xi_e=W_d/(2\ \pi\ F\ d)$, where: W_d energy dissipated in a single complete loading cycle and F maximum force reached by the isolation bearing in a single loading cycle. After fixing diameter ad height of the bearings (respectively 0.4 m and 0.2 m), it is possible to determine the shape factors S_1=A'/L and $S_2=D/t_e$ (where: A' common surface to the single elastomeric layer and to the single plate after deducting eventual holes, L free lateral surface of the single elastomeric layer increased by the lateral surface of the eventual holes, D dimension in plan of the single steel plate in parallel with the horizontal acting action and t_e total thickness of the elastomeric layers). The vertical pressure load acting on a single device is about of 6.0 MPa.

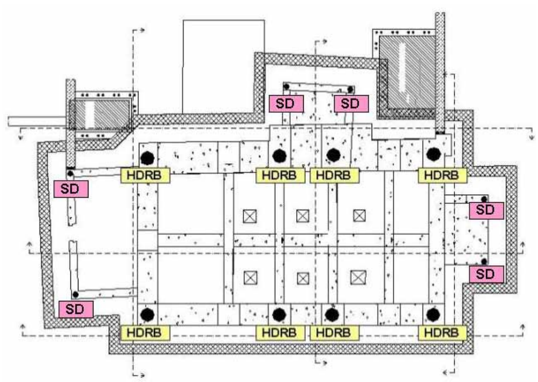


Fig. 13. Foundation string-course and disposition of HDRBs and SDs

Foundation and subfoundation

Complying with the conservation criteria, it has been decided to avoid vertical wall cuttings and insert the BI system providing a new r. c. foundation under the original masonry, taking into account the charnel-house presence under the pavement. The r. c. structure shall support the added bodies (sail bell-tower, sacristy and entrance stairs) and shall be rigid enough to take into account longitudinal and transversal slope, transferring SI actions to the ground. Sliding joints are obviously foreseen, in order to make completely free the building in the horizontal direction (respecting the design horizontal displacement) and avoid hammering effects. Fig. 14 shows the double foundation string-course, following the existing slope. The devices distribution in plan, as mentioned before, shall minimize torsional effects in case of earthquake. In a first phase, the construction of the superior string-course (connected to the church foundations) shall be provided; the inferior string-course (connected to the ground) shall be done in a second time. Between the string-courses, HDRB e DS devices have to be inserted with appropriate jacks. The inferior string-course execution steps are the following: reinforcement of the existing foundation walls, excavations through small sectors (about 50-80 cm of length) with adequate propping cribs, insertion of temporarily supporting joists, excavation for the realization of the string-course cast (sectors of about 150-200 cm of length) and laying of the r. c. reinforcing bars (Fig. 15). The masonry walls shall be connected to the subfoundation string-course by an appropriate anchorage (regular grid of inclined steel ties incorporated in the reinforced concrete). The temporarily supporting joist tips, left in the string-course, have to be cut at the end of the work. Due to the shallower foundation level of sail bell-tower with respect to the church nave, an additional integrating r. c. portion is foreseen. In order to permit inspection and maintenance of the BI system, a proper gap shall be realized, covered by a walking metallic grid anchored to the church walls (Fig. 16).

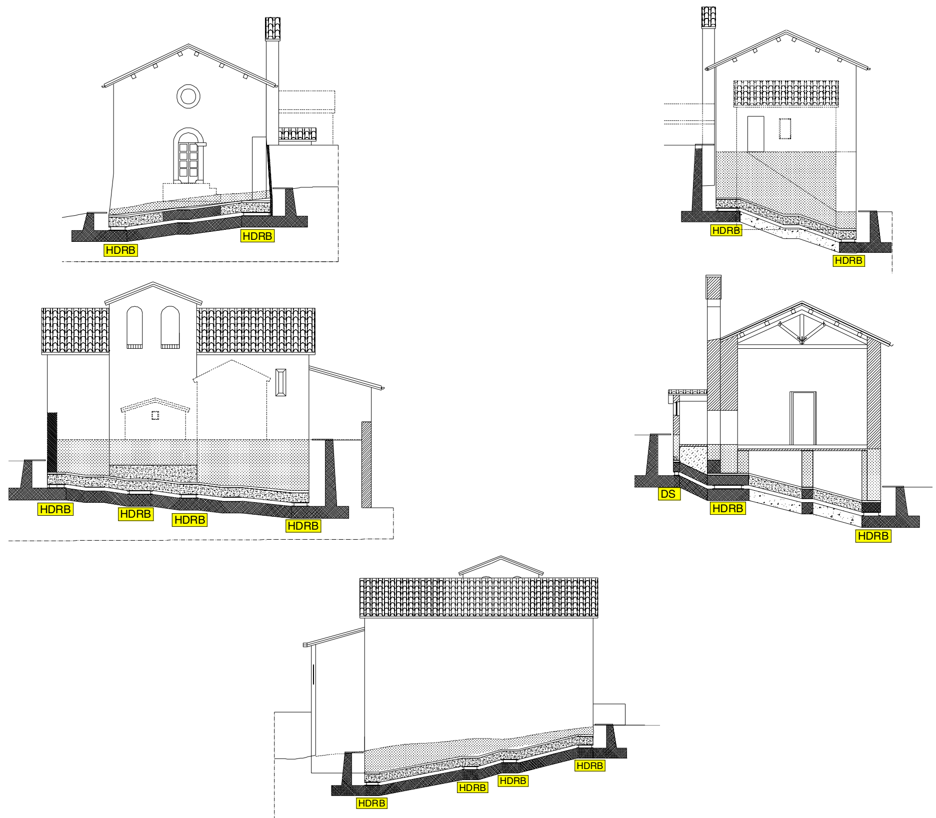


Fig. 14. Preliminary design of the subfoundation system

Fig. 15. Excavation for the string-course casting and laying of the r. c. reinforcing bars

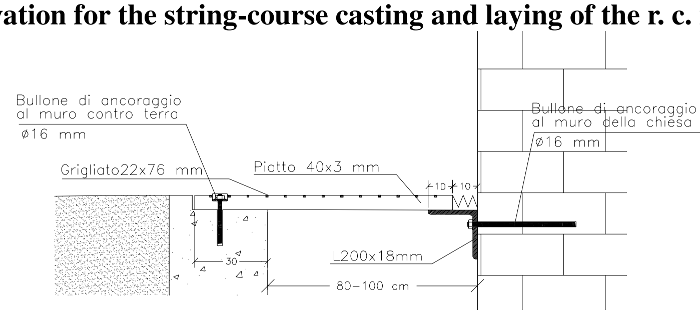


Fig. 16. Particular of the walking metallic grid for SI system inspection and maintenance

CONCLUSIONS

A seismic improvement of MCUHESs, if done with conventional methods, is often or ineffective or not compatible with conservation criteria. A clear example of this conflict is the case of the San Giovanni

Battista Romanesque church at Apagni (Sellano, Italy). Restored after the 1979 Valnerina earthquake but damaged again by the 1997 Marche-Umbria seismic event, it necessitated a new rehabilitation. Due to those simple statements, a different approach to the question has been pointed out, addressing to the use of modern antiseismic techniques; they can reduce the dynamic actions transmitted by the earthquake, rather than improve the structural resistance. After a detailed diagnostics and monitoring campaign, the church of San Giovanni Battista has been selected as a pilot application of Base Isolation (BI) by means of a specific sub-foundation system, in order to respect as better as possible the building original features.

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