Case Study: Seismic Retrofitting of a Medieval Bell Tower with FRP

Edoardo Cosenza[1] and Iunio Iervolino[2]

Abstract: Seismic retrofitting of monument structures requires compliance with restrictive constraints related to the preservation of original artistic and structural features. Any conceived intervention must achieve structural performance yet still respect the appearance and structural mechanism of the original and be as minimally invasive as possible. Therefore, traditional retrofit strategies may not be suitable for such purposes, and structural engineers need to develop specific techniques. Innovative materials (e.g., composites) may be helpful, as demonstrated by the case study presented in this paper. Fiber-reinforced plastics (FRPs) were used for the design, analysis, and installation of the retrofit for the medieval bell tower in Serra San Quirico (Ancona, Italy). A FRP tie system is applied to the inner walls and anchored at the base by a reinforced concrete slab, independent of the tower's foundation. The intervention enhances the seismic capacity of the structure and is fully provisional as it may be removed by heating the FRP with a hot air jet. The design process consisted of preliminary finite-element simulation and on-site structural assessment. Effectiveness is evaluated by a comparison of nonlinear static analyses (pushover) of the retrofitted and original structures. Finally, seismic risk reduction is computed by considering probabilistic seismic hazard at the site. Installation issues and the current appearance of the structure are also discussed.

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Introduction

Retrofitting existing structures to resist seismic actions that they were not originally designed for is a common practice in structural engineering. In Italy, for example, seismic retrofit is mandatory (because of the Friuli and Irpinia earthquakes in 1976 and 1980, respectively) in those regions where seismic hazard has been found. However, the topic may be very challenging if ancient constructions are considered. Their value, life expectancy, and safety margins (which have to be provided by retrofit) are different from those of ordinary constructions. These considerations require careful evaluation and possibly a strong and fruitful interaction between different competencies (Giangreco 2000; Penelis 2000).

The Charter of Venice (1964) and, more recently, the Charter of Krakow (2000) give comprehensive guidelines for the modern restoration of artistically relevant structures and may be considered the reference documents in the field. The basic principles are as follows: the interventions should have respect for the original materials; required replacements need to be harmoniously integrated with the whole, but easily identified; and additions are acceptable only if their influence on the other parts of the monument and/or its surroundings are negligible. In other words, the *minimum destruction theorem* applies. Moreover, any supplementary system should also be designed to be reversible—all the added components should be removable, leaving the structure as it was and allowing applications of new techniques with greater effectiveness. Also, because the typical life of such structures is much longer than that of ordinary buildings, common repair materials will most likely not have the necessary durability.

These working hypotheses are now widely accepted and regulated, especially in countries where these kinds of structures are a significant fraction of the built heritage. However, achieving seismic performance by interventions that respect the structural system and, at the same time, remain completely removable is often hardly possible. For this reason, the listed principles are intended, in general, as *asymptotic* concepts, meaning that they are targets not fully achievable by common technology. One can easily recognize that retrofit based on steel and reinforced concrete, which have been and are essential for structural restoration fitting common buildings, may not be suitable for structures belonging to the architectural and artistic heritage. Thus, innovative methods can be used to achieve the same or better performance than traditional approaches, while respecting the discussed principles. The case study concerning these issues includes design, analysis, and installation of the retrofit by fiber-reinforced plastics (FRPs) for a medieval bell tower. This project represents a good example of how composites may be used to accommodate some of the tradeoffs between structural and nonstructural demands that must be simultaneously satisfied.

[1]Full Professor, Dipartimento di Analisi e Progettazione Strutturale, Università degli Studi di Napoli Federico II, Via Claudio 21, 80125 Naples, Italy (corresponding author). E-mail: cosenza@unina.it

[2]Assistant Professor, Dipartimento di Analisi e Progettazione Strutturale, Università degli Studi di Napoli Federico II, Via Claudio 21, 80125 Naples, Italy. E-mail: iunio.iervolino@unina.it

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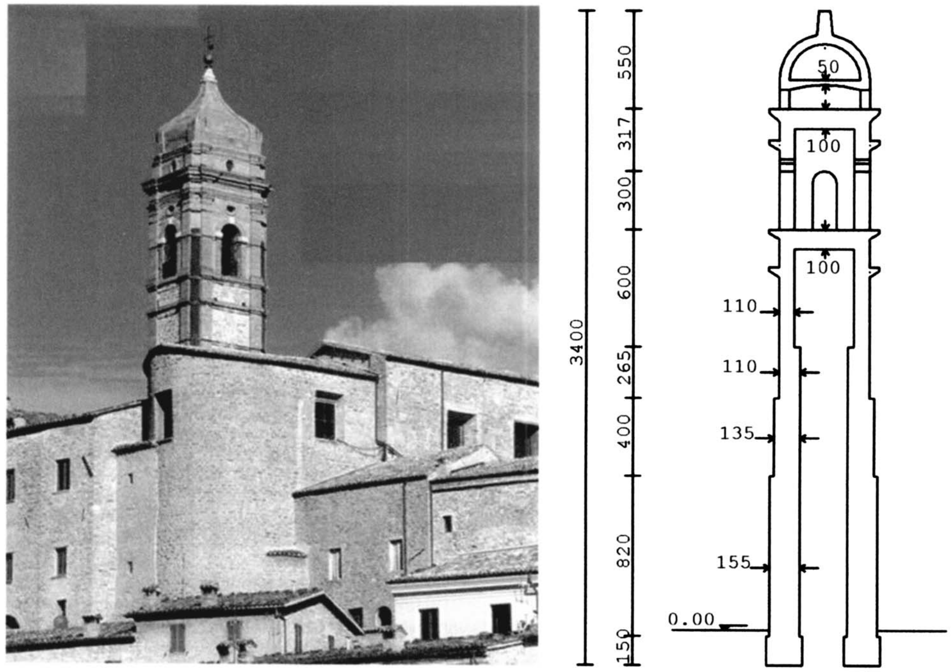

Fig. 1. Bell tower in Serra San Quirico after intervention and its longitudinal section (all measures in cm)

Santa Lucia's Church and Bell Tower

Affected by the Umbria–Marche earthquake (1997) (Mw 6), the bell tower of Santa Lucia's Church (Fig. 1) is a faced (multilayer) masonry structure built in the XVth century. It is located at the center of the little town of Serra San Quirico, a medieval suburb near Ancona, and is surrounded by many residential constructions of the same age. It is a calcareous masonry building, about 30 m in height and 1,200 tons in weight with a rectangular plan view. The wall thickness is 1.20 m at the basement and 0.80 m at the top. Along one direction the tower is connected to the vault roof of the church at about one-third of its height (see Fig. 5). The foundation is simply made of an augmentation of wall thickness 1.5 m underground. Nonstructural elements include wooden stairs and floors, the latter made of simply supported wooden boards, which should be preserved.

Because of damage and failure of similar structures in the same area, a desire to improve the seismic capacity of the tower was expressed by the local Architectural Heritage Supervision Office. Initially, to fulfill the scope of retrofitting, an intervention based on a steel reticular system anchored to the inner side of the tower was proposed by an engineering firm. The plan was to install a completely substitutive structure resistant to horizontal action in case of earthquake. The designed foundation for this system was a reinforced concrete slab 0.7 m thick, transmitting forces to the ground by 40 micropiles. Installation of such a system would require the anchoring of steel profiles in the masonry by appropriate devices and also the permanent removal of existing nonstructural elements such as floors, prescribing their replacement by steel panels. Access to the tower could be limited by this retrofit, and moreover, it could be potentially dangerous for the structure because of the stress concentrations in the masonry at anchoring points of steel profiles.

The architectural heritage authority recognized that this intervention violates the above-described principles and therefore rejected it (the foundation was already built at the time of rejection; this circumstance will be recalled later). The office then consulted the writers on the development of an alternative and acceptable solution. Innovative materials have been found helpful in the matter, and therefore a FRP intervention was designed, approved, and installed. This intervention enhances seismic capacity of the structure and is fully provisional, as discussed in the next sections. The design also included finite-element (FE) simulation and on-site structural assessment. Effectiveness of the intervention is herein evaluated by the comparison of nonlinear static analyses (pushover) of the retrofitted and original structures. Finally, seismic risk reduction is computed by considering probabilistic seismic hazard at the site. The whole process from conception to installation took only a few months, ending in spring 2001.

Numerical Structural Analysis and On-Site Dynamic Assessment

The information available on the static and dynamic conditions of the structure was poor at the time of consulting. Hence, to reach a preliminary knowledge, both numerical and on-site analyses were performed. The first step of investigation was material sampling to define the structure's properties. The faced masonry is made of two external layers of bricks filled by materials (like a sandwich). Its equivalent density was accurately estimated as 1,900 kg/m^3 by measuring the volume variation caused by samples immersed in water. It was not possible to evaluate other mechanical properties by the obtained samples; therefore, those have been assumed from the literature (Faella et al. 1993) and are given in

Table 1. Masonry Mechanical Properties

Specific weight (empirically measured)	Young's modulus	Transverse modulus
$\gamma = 1{,}900\ kg/m^3$	$E = 20{,}000\ kg/cm^2$	$G = 0.2\ E$

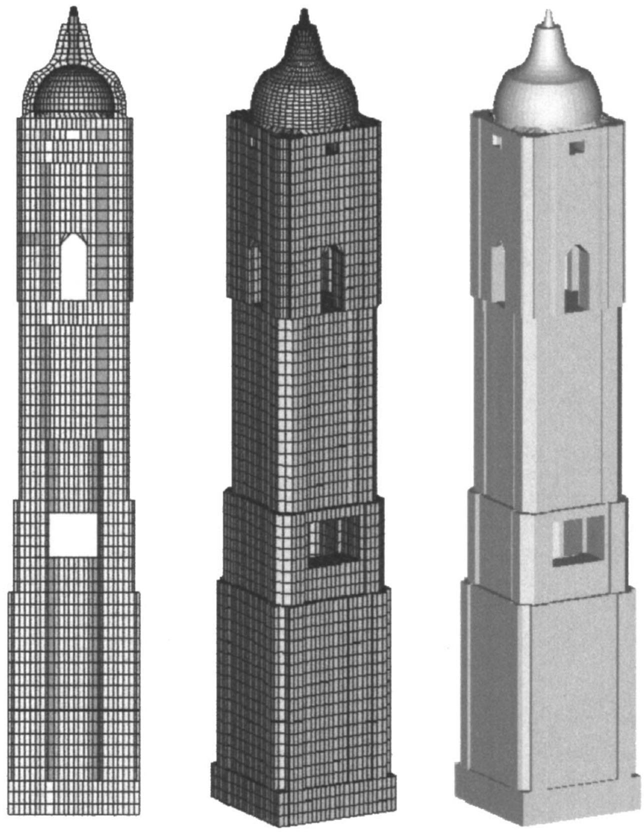

Fig. 2. 3D finite-element model and 3D rendering of structure

Table 1. Dynamic on-site assessment confirmed suitability of the assumptions.

To evaluate the dynamic properties and the stress levels (from self-weight and seismic horizontal actions) of the masonry in ideal conditions, a finite-element model (FEM) was developed on the basis of a detailed relief of the structure specifically commissioned. The 3D model, defined by 33,000 nodes and 27,000 8-node solid elements (Fig. 2), was developed with Altair Hypermesh software (http://www.altair.com).

As discussed, the tower is connected to the nearby building at 9 m from the first floor. The degree of constraint exerted by the connected church was uncertain, and therefore two FE subcases

Table 2. Finite-Element Results (Oscillation Frequencies and Periods) for Three Vibration Modes

Free subcase			Constrained subcase		
Mode	Frequency (Hz)	Period (s)	Mode	Frequency (Hz)	Period
1	1.10	0.91	1	2.27	0.44
2	1.13	0.88	2	2.40	0.41
3	4.86	0.20	3	7.21	0.14

were analyzed: (1) the tower is considered constrained at its base only, and hence it is assumed that the adjacent structure does not have any influence on the static mechanism; and (2) the cloister's roof-line connection is simulated by lateral constraints, not allowing horizontal translations while permitting rotations. It is recognized that the real condition is defined within these two cases, providing an upper and lower bound, respectively. The FE analysis results (in terms of deformed shape) for the first three vibration modes are given in Fig. 3. In Table 2, for the same modes, the oscillation frequencies and corresponding periods are given for the tower when it is not constrained by the roof of the church and when the constraint given by the adjacent structure is considered. The two different conditions result in very different shapes and periods, and consequently in different seismic forces. This numerical analysis also suggested that the tower's ideal state is adequate, since compression stress because of self-weight does not exceed 5 kg/cm^2 (maximum at the base of the tower), which is far below the estimated masonry strength (about 40 kg/cm^2).

Since the FE model provides only bounds to the real properties of the structure, an on-site structural assessment was conducted. The main aims of the experimental investigation were as follows: (1) dynamic characterization of the structure (determination of the actual natural frequencies and elastic properties, vibration damping, and modal shapes); (2) analysis of the degradation conditions by interpreting the dynamic behavior and comparing it with FE results; (3) validation of the numerical model; (4) verification of the structural behavior in real working conditions (e.g., dynamic response measurement in windy conditions); and (5) verification of the tower's constraint against the adjacent buildings. With all of this information, it was possible to properly estimate the seismic demand and create a correct modeling of the structure.

On-site dynamic assessment was performed by a unidirec-

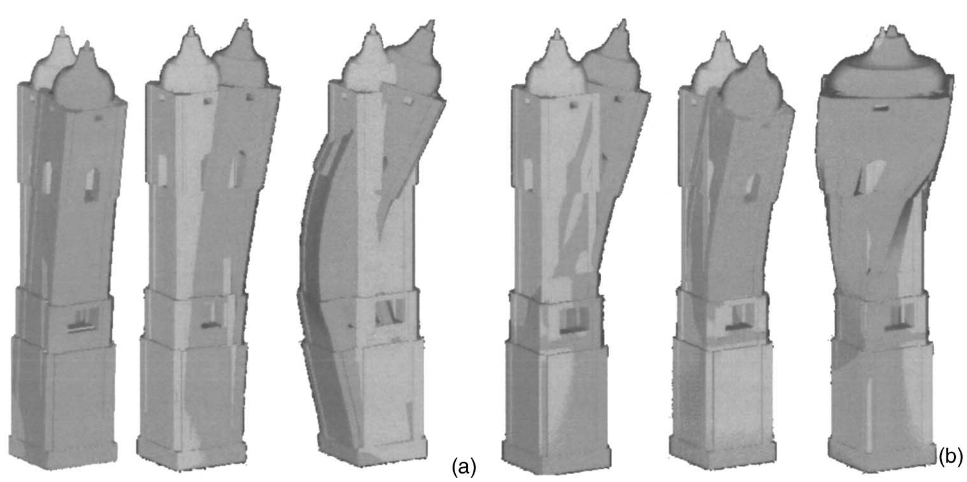

Fig. 3. Finite-element modal shapes for (a) free; (b) constrained cases

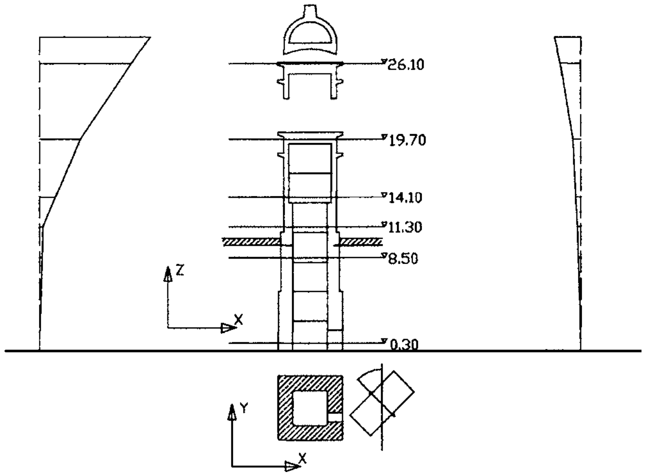

Fig. 4. Positions of accelerometers and vibrating machine (measures in m); experimental normalized first mode shapes in x- (right) and y-directions (left)

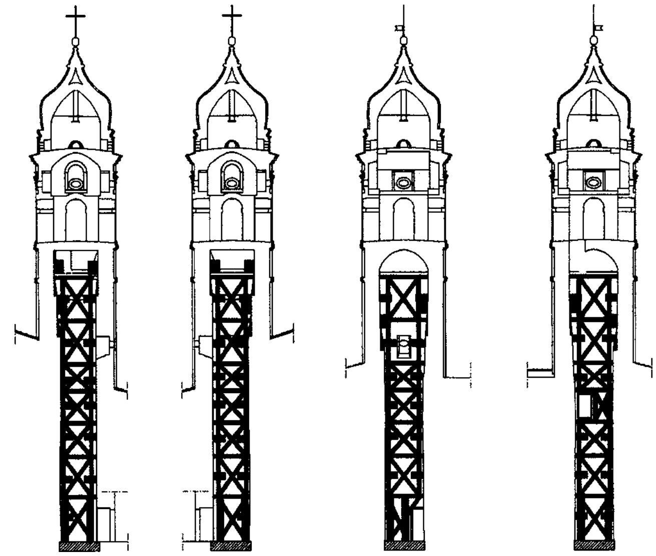

Fig. 5. Composite intervention relief

tional vibrating machine placed at the base of the structure with a 45° angle in respect to the plan view of the tower to excite both x- and y-directions at the same time. For the structural response measurements, 16 accelerometers positioned across two perpendicular directions at different heights (Fig. 4) were used. Inducing strong vibrations in this kind of structure can be dangerous, sometimes destroying what should be preserved, and therefore, very sensitive instruments were employed. These instruments can measure acceleration as low as 10 μg to identify the tower's natural frequencies by moderate dynamic forces, which do not induce any damage to the structure or to the adjacent buildings. The first two observed frequencies (Table 3) are 1.95 and 2.20 Hz. These values are bounded by those obtained in the 3D FE analyses considering the tower to be constrained only at its base (1.10 and 1.13 Hz) and closely related to the interaction with the adjacent church (2.27 and 2.40 Hz). The real constraint level is closer to the latter FE simulation rather than the former.

The experimental structural identification also shows that the natural frequency (corresponding to a bending mode) in the x-direction is lower than in the y-direction (Fig. 4). This happens because of a lower bending stiffness in the x-direction and/or because of the different constraint conditions. In fact, in the displacement diagram, a slope change happens at the bell tower's junction with the rest of the structure, which demonstrates a good connection with the church. The reliability of results from dynamic and FE analyses is established by the common identification of a third torsional mode. Because of the similarity of the numerical (ideal) and on-site data, it was also possible to conclude that no major damage or degradation were present in the structure. However, as described in the following paragraphs, the structure does not withstand the seismic demand at the site, and therefore the retrofit is still needed to reduce the seismic risk. The experiment also suggested that from the static point of view, the bell tower is in a condition where the induced excitation, if moderate, is partially dispersed by the church and cloister. Finally, note that the test also proved that the accelerometers' sensitivity and accuracy led to the dynamic identification (performed with the traffic, environmental, or wind vibrations), producing exactly the same result as the analysis by the vibrating machine. These circumstances make this kind of test very interesting for the safe on-site analysis of monument buildings and other critical structures.

Table 3. On-Site Dynamic Assessment Results

Mode	Frequency (Hz)	Period (s)	Damping (%)	Oscillation type
1	1.95	0.51	2.28	1° bending X
2	2.20	0.45	1.76	1° bending Y
3	6.75	0.15	2.05	Torsional

FRP Retrofit

The retrofit conception process was intended to satisfy the targets given in the Introduction of being as effective and transparent as possible. The proposed intervention aims to apply, by appropriate techniques, a reticular system made of vertical, horizontal, and diagonal FRP sheets adherent to the masonry. Fig. 5 shows the FRP system as it appears on the four inner walls of the structures. Mapewrap carbon fibers (600 g/m^2) 0.335 mm in thickness (t) and 20 cm in width (w), produced by Mapei (http://www.mapei.it/), were employed. To improve the bonding, additional horizontal short composite elements were positioned in the corners of the walls, and at the first floor the vertical elements are 40 cm wide. The ultimate tensile strength (σ) of the fibers is 48,000 kg/cm^2, then considering the four vertical elements plus four diagonal ($\alpha = 45°$) with an efficiency factor of 0.7, the total tensile force (F_u) the composite system may sustain (linear stress–strain behavior) is given by Eq. (1)

$$F_u = 4[1 + \cos(\alpha)]\sigma \cdot w \cdot t = 219 \text{ t} \tag{1}$$

Usually structural engineering practice neglects the tension strength of masonry, while FRP ensures a monolithic behavior for high-intensity earthquakes. Since each base wall, if separated from the rest of the structure, is about 475 t in weight, the masonry at the base would not experience any tension stress when the FRP is at the ultimate force.

With this intervention the structure retains its static mechanism for service loads and for low-intensity seismic activity because the added stiffness is negligible in comparison to that of the bell tower, but in the case of strong motion, the side of masonry in

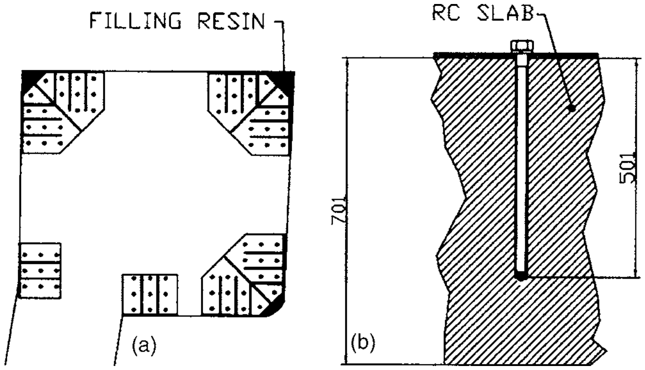

Fig. 6. (a) Base anchoring plan view; (b) anchoring system detail

tension loses cohesion and triggers tension loading of the composites. Therefore, the masonry behaves as a reinforced structure with the FRP-absorbing tension.

From the structural point of view, FRP improves the seismic capacity of the tower, as demonstrated by the nonlinear analyses (to follow), but this kind of intervention has several other advantages: (1) it is reversible since the FRP may be removed by thermal nondestructive procedures; (2) the FRP reticular systems do not change the main structural mechanism because the composites are engaged only when exceptional loads are applied, providing extra tension strength to the system; (3) the system does not develop nodal forces because collaboration with masonry is spatially continuous; and (4) the preexisting foundation made of a 70-cm thick reinforced concrete slab and 40 micropiles at the base of the structure (built as the foundation for the rejected steel intervention) allows an adequate anchoring of the composites and transfer of tensile forces to the ground without overloading the bell tower. This is possible because the foundation is independent from the structure.

Key aspects of the intervention are the composite-masonry bond, the fabrication details, and the anchoring of the composites to the foundation. In fact, many issues regarding the bond of the FRP system to the substrate remain the focus of a great deal of research (Cosenza et al. 2000; Kiss et al. 2002; Ceroni et al. 2003; Aiello and Sciolti 2006). For both flexural and shear strengthening, many different varieties of debonding failures can govern the strength of a FRP-strengthened structural subsystem. Throughout the design procedures, significant limitations on the strain level achieved in the FRP material are imposed to conservatively account for debonding failure modes. The reticular pattern of the FRP reinforcement adopted in this case (Fig. 5) prevents *end plate mode debonding*, while the *intermediate mode debonding* is limited by the moderate working stress of FRP reinforcement. However, to obtain a good bond, masonry samples were analyzed to optimize the surface treatment, which consisted of scarification of the interface (Balsamo et al. 2001).

At the base, four stainless steel plates [Fig. 6(a)] are fixed to the composite with resin. The contact area is 40 by 40 cm for each element. Anchoring plates are designed to be stiff because they should not be the weakest element of the system. The bottom of those plates is anchored into the RC slab by special stainless steel bolts [Fig. 6(b)], which were designed to transfer the ultimate force the FRP can sustain. A Hilti (http://www.hilti.com) anchoring system was used to achieve this target. In particular, expansive resins, injected into the drilled bolt holes, aid in transferring forces to concrete.

The durability and long-term performance of FRP materials are also the subject of ongoing research. Long-term field data are not currently available, and the life of FRP-strengthening systems is difficult to accurately predict. Usually, long-term fatigue and creep are addressed by stress limitations. ACI-440 (ACI 2002), which is specific for FRP retrofit of concrete structures, recommends investigating the effects of a variety of environmental conditions such as steel corrosion, silica aggregate reactions, and water entrapment. All these issues may not apply to the case under examination because the masonry is not subjected to the chemical reactions of the concrete. Moreover, the FRP system is designed to be unloaded in normal (nonseismic) conditions. Therefore, coupling effects of loading and environment do not take place.

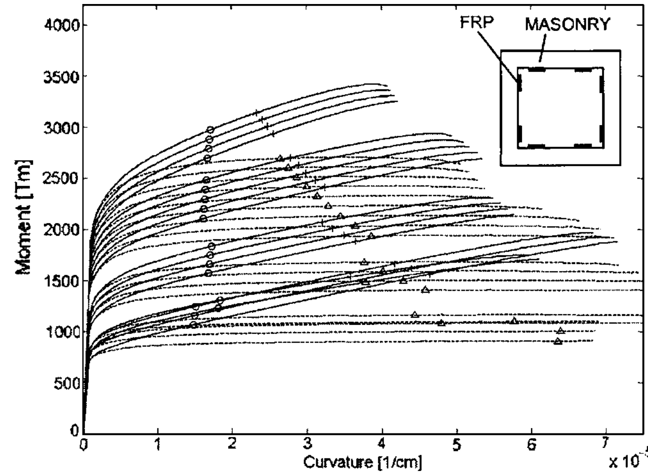

Fig. 7. Retrofitted (solid) and unretrofitted (dotted) sections' moment–curvature diagrams and scheme of typical reinforced cross section

FRP is applied to the inner walls of the structure and is therefore not exposed to UV rays that may affect the aging of the FRP matrix. The surface treatment by sand (see section on installation) further protects the fibers. Therefore in this case long-term and environmental effects are not relevant to the structural performance. The factors associated with the long-term durability of the FRP system do not influence the tensile modulus of the material used for design since, generally, the tensile modulus of FRP materials is not affected by environmental conditions. However, instrumental monitoring every 5 years was programmed to verify the FRP status.

Nonlinear Analysis

Nonlinear analysis was necessary to properly evaluate the FRP contribution to the behavior of the tower. In particular, a pushover (PO) analysis was developed to compare the seismic behavior of the structure before and after the intervention. The tower was analyzed by a specifically developed *distributed plasticity* model—the structure is divided into a series of contiguous subelements with constant stiffness and characterized by a specific nonlinear moment–curvature (MC) relationship. The model of the tower consists of 18 subelements, capturing all variations in geometry and vertical load along the height of the structure. MC diagrams were computed for both the reinforced and unreinforced cases (Fig. 7). The masonry stress–strain relationship was modeled by the Powell and Hodgkinson (1976) constitutive law—the

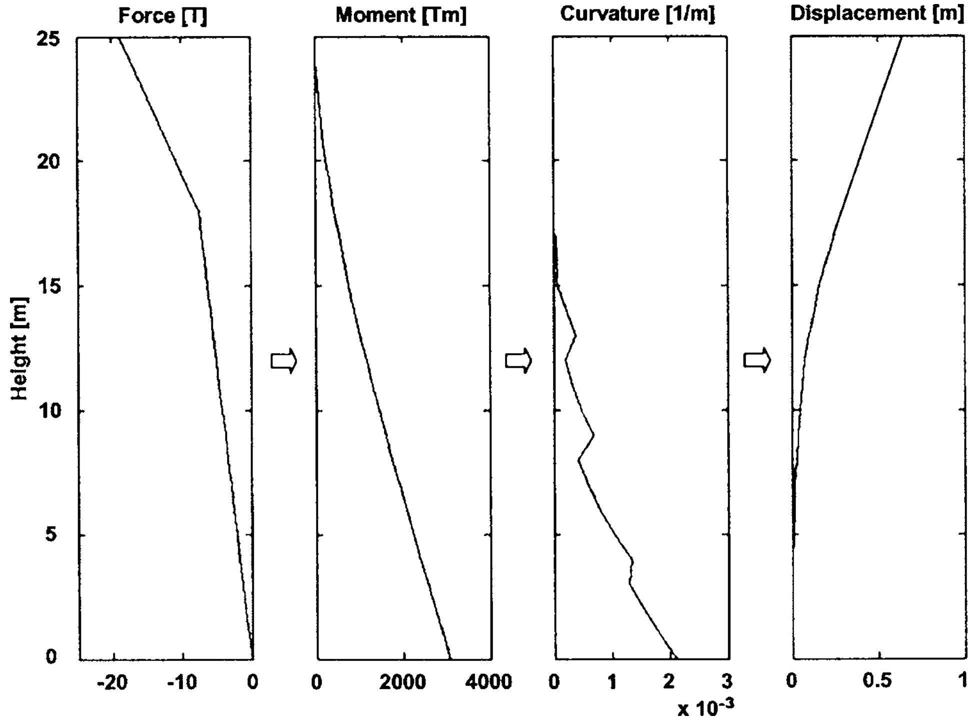

Fig. 8. Scheme of pushover analysis

peak deformation was 0.2% and ultimate deformation was 0.4%. The masonry was assumed to sustain no tension, and FRP has been considered linear-elastic in tension and not contributing to compression. Other mechanical assumptions were the equal deformation of FRP and masonry and that sections remained plane after deformation. Circles on the MC curves of Fig. 7 correspond to a 0.5% FRP deformation, while crosses and triangles refer to a 0.2% deformation of the masonry.

The distribution of horizontal forces used to compute the PO curve corresponds to the first oscillation mode of the structure free from the church. This conservative hypothesis assumes that in the case of a high-intensity seismic event the constraint made by the church becomes ineffective. Therefore in the PO the structure is considered to be isolated with respect to the other buildings. The load–displacement curve was computed by numerical integration of the curvatures as in the scheme of Fig. 8 where a generic step of the PO analysis is represented as an example. Monitored displacement is that of the last reinforced section at 18 m from the ground, which is approximately two-thirds of the total height of the structure.

PO curves for the retrofitted and unretrofitted structures are given in Fig. 9, in which a *shear* limit state is considered. In fact, for the unretrofitted structure, the interaction of bending and shear has to be taken into account. The Mann and Müller model (1982) has been considered to include this failure mode as it reflects three limit states for the masonry: compression failure, shear failure (Mohr–Coulomb mechanism), and tension failure. The resulting bending–shear stress interaction domain is given in Fig. 10. Since the unretrofitted structure's real capacity is bounded by limit states of maximum masonry deformation and shear failure, both conditions are reported on the corresponding curve.

The PO analysis referring to the retrofitted case reports both limit states because of masonry (0.2%) and FRP (0.5%) deformations. Shear limit state is not considered in this case because it is assumed that shear is all taken by the tension of vertical and diagonal elements of the FRP system. In fact, at collapse, the FRP reticular system transfers shear by horizontal elements, which behave as stirrups, and diagonal elements, which are necessary to reach equilibrium. Therefore, no significant shear stress is generated in the masonry. This lack of shear stress is one of the main benefits of this application.

The effectiveness of the intervention can be preliminarily recognized by direct comparison of the two curves in Fig. 9; the capacity of the structure is enhanced independently of the limit state considered. The reticular composite system does not change the elastic behavior and the base-shear strength. On the other hand, by providing additional tensile capacity, the FRP system increases the maximum displacement the structure can undergo.

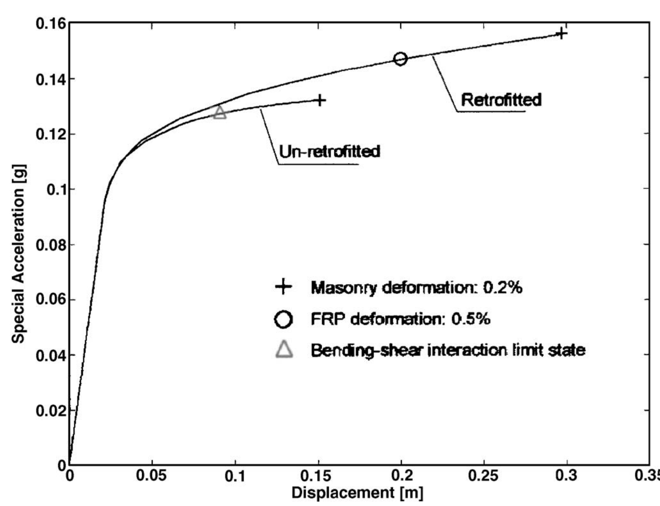

Fig. 9. Retrofitted and unretrofitted pushover curve comparison

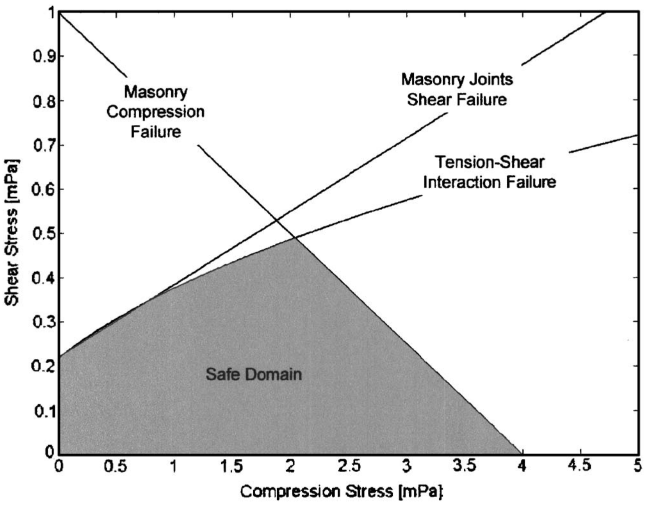

Fig. 10. Bending-shear interaction domain

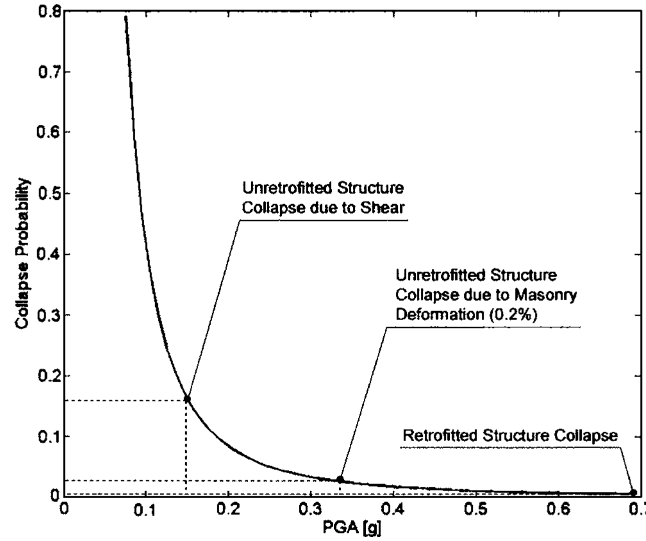

Fig. 12. Hazard curve for Serra San Quirico

Seismic Risk Reduction

The PO analysis makes it possible to compare capacity to seismic demand at the site using the *Capacity Spectrum Method* (Fajfar 1999). This method allows a picture, on the same plane, of the nonlinear capacity and the inelastic spectral demand. The seismic *performance point* of the structure is defined as the intersection of the inelastic spectrum and the bilinear capacity curve retrieved by the PO analysis. To this aim, the spectrum has to be represented as an acceleration displacement response spectrum (ADRS). For this case study, the elastic spectrum, according to Italian seismic code (OPCM 3274 2003) for the seismic level of the Serra San Quirico area, has a peak ground acceleration (PGA) equal to 0.25 *g*. This PGA has to be amplified by a factor of 1.25, taking into account soil conditions and by a factor of 1.4 taking into account the importance of the structure, which is located in the residential part of the town and is right above the town-hall building. In Fig. 11 the elastic, in its ADRS format, and the inelastic constant ductility (4.5) spectra are given together with the capacity curves for the two limit states of the unretrofitted structure.

As prescribed by the method, the capacity curve has been rendered bilinear for any of the limit states. Therefore, two different bilinear curves have been retrieved, each of which has the last point coincident with a specific ultimate condition. This action allows equivalence with an elastic–-plastic, single-degree-of-freedom (SDOF) system for each failure mode. Then the performance points can be retrieved. It is possible to see the points for the masonry deformation limit state; however, the bilinear curve for the shear limit state does not intersect the inelastic spectrum, and therefore the unretrofitted structure does not withstand the seismic demand in this case (e.g., unreinforced capacity is lower than the demand if the shear effect is considered). For the case of reinforced structure, not reported in the plot for sake of brevity, capacity largely exceeds demand.

The seismic risk reduction (Fig. 12) induced by FRP retrofit is obtained by first computing the maximum PGA the structure can sustain. It is the PGA of the elastic spectrum that gives a performance point coincident with the collapse point of the PO curve. Second, the exceeding probability associated with this PGA (and therefore the collapse probability or seismic risk) is retrieved by the probabilistic hazard curve at the site. Fig. 12 shows the hazard curve for Serra San Quirico (SSN 2001), as well as the collapse PGAs for both the unretrofitted (referring to the two possible limit states) and the retrofitted structures. This risk analysis predicts a relatively high failure probability for the unreinforced structure, while the retrofitted design increases the safety margin by reducing the probability of the ground motion causing collapse to a value compatible with the historic and artistic relevance of the bell tower.

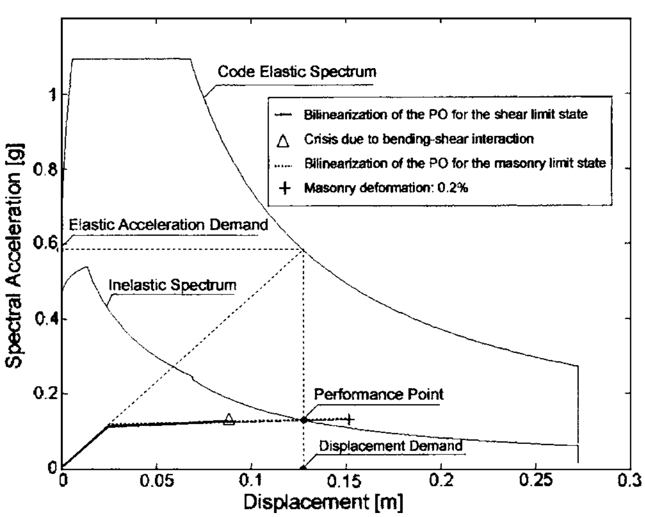

Fig. 11. Unreinforced structure capacity spectrum

Installation

Installation of composites strictly followed the design specifications. Even though the small available space and the presence of the nonstructural elements were constraining factors, the composite allowed a simple application and implementation of the intervention. The FRP was installed without removing the original wooden beams, while floors, which are made of wooden boards simply supported by the mentioned beams, were temporarily removed and put in place again at the end. The composite's reticular system geometry was also locally modified to not pass over the tower openings. Fig. 13 highlights those aspects where the an-

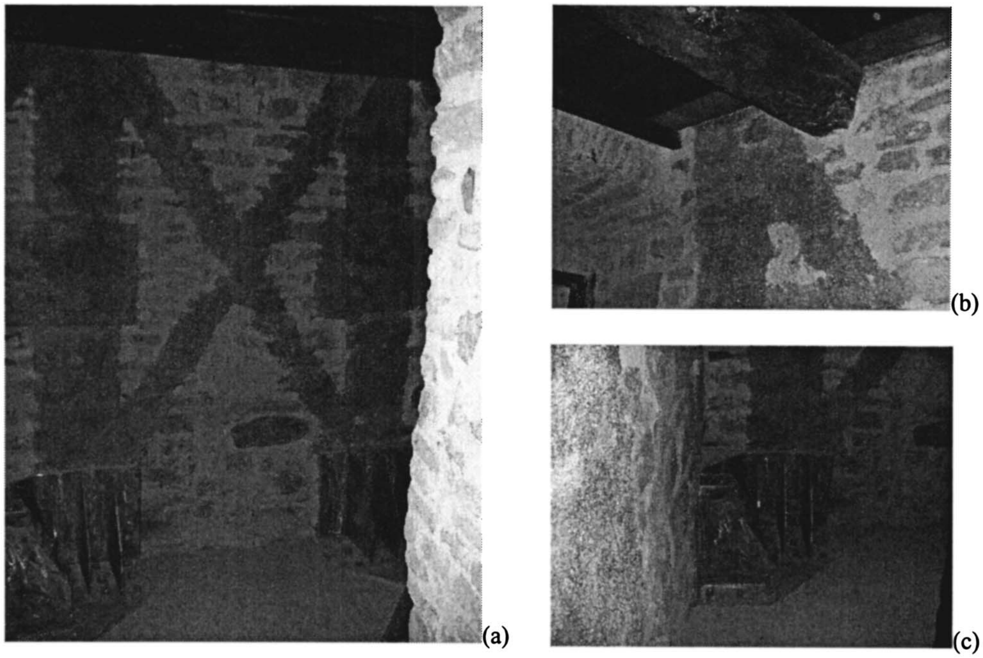

Fig. 13. (a) Installed composites and base anchorage; (b) composites near the unremoved wooden floor; and (c) intervention anchoring details

choring steel plate detail is shown, and the horizontal and oblique elements are displayed close to an original beam. The lack of brightness in the photos is because of the achievement of the "transparency" target by a surface treatment with sand, which gives the FRP an appearance more compatible with the original masonry.

Note that the intervention is to be considered "reversible" because the applied FRP can be removed by an air jet. In fact, in a lab test, composite material was heated by an air furnace, while monitoring temperatures of FRP and underlying brick. At an air temperature of about 300°C, the resin had a second-order transition, which is similar to melting (temperature of the resin about 90°C), and the composite was removed easily (Guglielmo and Cosenza 2003). The brick temperature was about the same as that of the surrounding environment, meaning that thermal inertia of the masonry is sufficient to preserve art works that may be on the other side of the wall. Since the results only reflect global behavior of the structure, several local retrofit interventions along the structure were delivered to avoid local collapses, which are quite common in this kind of structure. Following this concern, traditional improvements of the masonry and steel chains were also applied to the bell's room at the top of the tower.

Conclusions

The intervention on the bell tower of Santa Lucia's Church is an application of composite materials for the seismic retrofit of historic monuments where traditional retrofit strategies do not likely apply. Priority targets of transparency, low impact, and structural effectiveness have been fully achieved even if they are commonly considered asymptotic. Nonlinear analysis and seismic risk evaluation by capacity spectrum show that the retrofit minimizes the failure probability if global reinforcement is accompanied by local traditional interventions on masonry. The designed system is also in full compliance with the other requirements because it

- Is not a substitute for the original structure but complementary strengthening action for the masonry;
- Is barely visible and fully reversible since the FRP may be removed with nondestructive techniques;
- Does not concentrate forces since interaction with masonry is continuous (avoiding local failures); and
- Has appropriate durability although it requires periodic monitoring.

This study is also a case of fruitful interaction between state offices, architects, and engineers through innovative structural techniques.

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References

Aiello, M. A., and Sciolti, S. M. (2006). "Bond analysis of masonry structures strengthened with CFRP sheets." *AI Comm.*, 20(1–2), 90–100.

American Concrete Institute (ACI). (2002). "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures." *Technical Committee Document 440. 2R-02*, Farmington Hills, Mich.

Balsamo, A., Battista, U., Herzalla, A., and Viskovic, A. (2001). "The use of aramidic fibres to improve the structural behaviour of masonry structures under seismic actions." *Proc., Archi 2000 Int. Congress*, UNESCO, Paris.

Ceroni, F., Pecce, M., Manfredi, G., and Marcari, G. (2003). "Experimental bond behaviour in masonry elements externally reinforced with FRP laminates." *Proc., CCC2003 Composite in Construction*, Università della Calabria, Cosenza, Italy.

Charter of Krakow. (2000). "Principles for conservation and restoration of built heritage." *Proc., Int. Conf. Conservation "Krakow 2000*," ICOMOS, Paris.

Charter of Venice. (1964). "International charter for the conservation and restoration of monuments and sites. Decisions and resolutions." *Proc., 2nd Int. Conf. of Architects and Technicians of Historical Monuments*, ICOMOS, Paris.

Cosenza, E., Manfredi, G., Occhiuzzi, A., and Pecce, M. (2000). "Toward the investigation of the interface behavior between tuff masonry and FRP fabrics." *Proc., Int. Conf. on Mechanics of Masonry Structures Strengthened with FRP Materials: Modeling, Testing Design, Control*, Edizioni Cortina, Padova, Italy.

Faella, G., Manfredi, G., and Realfonzo, R. (1993). "Stress-strain relationships for tufaff-masonry: Experimental results and analytical formulations." *Masonry Int.*, 7(2), 35–61.

Fajfar, P. (1999). "Capacity spectrum method based on inelastic demand spectra." *Earthquake Eng. Struct. Dyn.* 28(9), 979–999.

Giangreco, E. (2000). "Principles of structural restoration, developments and perspectives." *Proc., 5th Int. Congress on Restoration of Architectural Heritage "Firenze 2000*," CICOP, Tenerife, Spain.

Guglielmo, E., and Cosenza, G. (2003). "Un esempio di miglioramento sismico reversibile: Il caso del campanile della chiesa di S. Lucia." *Proc., 19th Int. Conf. of Science and Culture; La reversibilita net restauro: Riflessioni, Esperienze, Precorsi di ricerca*, Arcadia Ricerche, Bressanone, Italy (in Italian).

Italian Seismic Survey (SSN). (2001). *Seismic hazard for the Italian territory* (CD-ROM), Rome (in Italian).

Kiss, R. M., Kollar, L. P., Jai, J., and Krawinkler, H. (2002). "Masonry strengthened with FRP subjected to combined bending and compression. II: Test results and model predictions." *J. Compos. Mater.*, 36(9), 1049–1063.

Mann, W., and Müller, H. (1982). "Failure of shear-stressed masonry—An enlarged theory, tests and application to shear walls." *Proc., British Ceramic Society*, 30, 223–235.

Ordinanza del Presidente del Consiglio dei Ministri (OPCM) n. 3274. (2003). "Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e normative tecniche per le costruzioni in zona sismica." *Supplemento ordinario 72 alla Gazzetta Ufficiale della Repubblica Italiana n° 105* (in Italian).

Penelis, G. G. (2000). "Analysis and design in structural restoration." *Proc., 5th Int. Congress on Restoration of Architectural Heritage "Firenze 2000*." CICOP, Tenerife, Spain.

Powell, B., and Hodgkinson, H. R. (1976). "The determination of stress/strain relationship of brickwork." *Proc., 4IBMAC*, Groupement National de l'Industrie de la Terre Cuite, Brugge, Belgium.