Modelling and analysis of a Romanesque church under earthquake loading: Assessment of seismic resistance

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Abstract

In this paper a Romanesque masonry church is analysed in order to assess its structural behaviour and its seismic vulnerability with respect to the actual state of conservation. Starting from a specific case study, a contribution to the issue of modelling and analysis of monumental masonry buildings under seismic action is provided. A finite element methodology for the static and dynamic nonlinear analysis of historical masonry structures is described and applied to the case study. A quasi-static approach (the seismic coefficient method) for the evaluation of the seismic loads has been used (as indeed is common in many analyses of the seismic behaviour of masonry structures). The comparison demand vs. capacity confirms the susceptibility of this type of building to extensive damage and possibly to collapse, as frequently observed. Moreover the actual efficiency of current techniques for repairing and strengthening are analyzed in order to evaluate their benefits. The analysis of repairing and strengthening techniques show the effectiveness of the usual structural reinforcement in terms of increased seismic capacity. The paper advocates that significant information can be obtained from advanced numerical analysis, namely with respect to the understanding of existing damage and to the minimum and adequate design of strengthening. A clear understanding of the structural behaviour and reliable strengthening, based on sophisticated tools of structural analysis, can therefore reduce the extent of the remedial measures in the restoration of ancient structures.

Keywords: Romanesque church buildings; Earthquake loading; Nonlinear analysis; FE modelling; Seismic vulnerability; Strengthening techniques

1. Introduction

A large portion of the Italian cultural heritage is provided by Church masonry building. These historical buildings have demonstrated during the past to be particularly susceptible to damage, and prone to partial or total collapse, under earthquake loads, sometimes due to non-respectful restoration [1,2]. As a matter of fact repairs and retrofitting techniques should always respect the original existence; any intervention not respectful of it could also create incompatibility with the original structural behaviour.

Masonry buildings are generally able to carry the vertical loads in a very safe and stable way, while they are rather sensitive, from a structural point of view, to horizontal loads. The high seismic vulnerability of this type of building is due both to the particular configuration (often characterized by open space, slender walls, lack of effective connections among the structural elements) and to the mechanical properties of the masonry material (highly nonlinear behaviour and very small tensile strength). If in principle, the prediction of the structural response of monumental buildings is not different from that of other constructed facilities (e.g. a bridge) it is an even more challenging task for several reasons [3–5].

Each monumental building is by definition a unique building characterised by its own history, often resulting in a composite mixture of added or substituted structural elements, strongly interacting; the dynamic (and static, for that matter) behaviour of ancient buildings is normally too complicated to be interpreted by simple mechanical models. In particular trying to extrapolate analytical procedures specifically developed for modern buildings is in most cases inadequate, since the static diagram is substantially different from the one of modern structures made of trusses and frames. Moreover it is quite...
difficult to perform reliable quantitative strength evaluations, due to the difficulty of gathering experimental data on the resistance of the structural elements and even on the mechanical properties of the materials on site [6–8]. Restrictions in the possibility to inspect the construction, or difficulties on the removal of specimens in buildings of historical value (as well as the high costs involved in inspection and diagnosis) often result in limited information about the internal constructive system or the properties of existing materials. Moreover, another aspect that’s important to take into account, is that structural resistance of material decreases in time due to deterioration, and this degradation is frequently accelerated by neglect or carelessness.

In brief, monumental historical buildings are by definition buildings that it’s difficult to reduce to any standard structural scheme because of the uncertainties that affect the structural behaviour and mechanical properties. The above considerations explain the need of specific modelling and analysis strategies for historic masonry constructions.

In this paper, starting from a single case study, a contribution to the issue of modelling and analysis of Romanesque churches under seismic action is provided: a relevant case study that demonstrates the careful use of numerical analyses to face practical engineering problems in the field of historical construction is presented.

An evaluation of the capacity of the church to withstand lateral loads together with the expected demands from seismic actions is also given. The effects of the current techniques for repairing and strengthening are then investigated in order to evaluate the effectiveness of the usual retrofitting techniques.

2. The object of study

2.1. Description of the structure

This paper addresses the concern of seismic analysis and vulnerability of Romanesque churches with respect to a specific case study: the Farneta abbey (see Fig. 1). This church is located in Val di Chiana (Cortona, Italy). The plan view shows the typical basilica layout with the main nave, the clerestory transept and a triple apse (see Fig. 2). The main dimensions were a maximum length of about 26.5 m, a maximum width of 21.5 m and a wall height of about 11.0 m. The masonry walls’ thickness, approximately, varies between 0.70 m (main nave walls) and 0.90 m (clerestory transept walls), see Figs. 3 and 4. At the left and right side of the clerestory, by two stairs, it is possible to go down to a crypt that is entirely developed under the clerestory area (see Fig. 4). The roof of the church (main nave and clerestory transept) is made with a timber structure. Crypt roof is made by a mixture of stone and brick vaults connected with the confining walls and sustained by two granite circular columns. With its typical layout the Farneta abbey could be considered representative of a wide class of Romanesque churches existing in central Italy.

The original Romanesque structure was built between 700 and 800 AD by the Ronzano’s Count. The period of maximum majesty of the abbey was from the tenth to the thirteenth century, under the Benedictine monks. After this time Farneta lost its autonomy (both civic then ecclesiastic). It will be necessary to attend the eighteenth century so that Farneta repurchases its autonomy. Today the abbey is under the ecclesiastical jurisdiction of Cortona [9]. Now it is possible to understand from its history various additions and alterations which followed during the centuries, and the original Romanesque structure was quite different from what is seen today. The main nave, for instance, was longer than the actual fourteen meters. This reduction was made during the eighteenth century when, probably, the construction became tumbledown and needed a restructuring.

Several types of masonry weaving are present along the wall of the church. They are different both for material (stone, brick, etc.) and for shape (“opus incertum”, “opus mixtum”, “opus quadratum”, etc.). This variegated picture reflects the modifications endured over the centuries by the building. However, despite these differences on the masonry texture, the construction is mostly made of irregular sandstone masonry (the local stone) with thick lime mortar joints, that is used in general (see Figs. 5–7). Different masonry types are, namely: (a) irregular sandstone masonry with disordered brick (see Fig. 5); (b) regular stone with thin lime mortar joints used on the areas after the eighteenth-century restructuring; (c) brick masonry for the crypt’s vaults (see Fig. 8). Local stone is used also as cladding for windows and doors. In order to have a mechanical characterization of these masonry types several different techniques can be used, both static (flat-jack testing
Fig. 3. Plan layout of the church case study (meters).

Fig. 4. Plan layout of the crypt (meters).
and coring) and dynamic [10,11]. For the following analyses the mechanical properties of the Farneta materials were agreed with the data on existing tests of similar masonry and in situ testing [12,11]. Substantially information was already available from similar materials and the conservative values could be estimated from this experience: typical values for old stone masonry were selected for the material parameters.

The last restoration on the church was done in 1964 when, in the area around the apse, a new foundation was built, modifying the hydraulic tenor of soil. At the same time the crypt that was full of material accumulated during the centuries was emptied. After this restoration works along the northern wall of the transept (close to the apse) the church exhibited a variegated cracking pattern (see Figs. 9–11).
2.2. Actual state of conservation

In 1964 the last restoration on the church was done. The eighteenth-century shape of the church was eliminated, and the original Romanesque form was recovered. From a structural point of view this work concerned principally the apse and the area around it. In this restoration the crypt under the apse, that was full of material (and also full of water), was completely emptied. Around the apse, in the external area of the church, a new foundation was built modifying the hydraulic tenor of soil [9].

After these works, along the northern wall of the transept (close to the apse) the abbey exhibits a variegated cracking pattern along the transept and the apse. Figs. 10 and 11 shows the variegated cracking picture around the apse and the transept that is the result, in the authors’ hypothesis, of two different mechanisms. A first mechanism is the movement and outside rotation of the apse area (confirmed by the presence of full crack along the transept). This is due to the ground consolidation following the work around the external area of the apse (see Fig. 12). The second one is the movement of northern and southern walls; this is due, certainly, to the presence of the thrust of both the main arch of the nave and the apses (Fig. 13).

The knowledge of the actual cracking path, and then the state of conservation of the monument, is a crucial task for the assessment of the church’s vulnerability under earthquake loads.

3. Analysis method

Seismic evaluation methods need the fundamental dynamic properties of the church. In this paper, the dynamic properties needed are obtained by using the finite element method. A 3D linear and nonlinear analysis that take into account the
nonlinear behaviour of masonry are performed: constitutive assumptions, characterized by elasticity, damage and friction, are done. The behaviour of the masonry is replicated by use of a solid element that can have its stiffness modified by the development of cracking and crushing. The standard FEM modelling strategy is used based on the concepts of homogenized material and smeared cracking constitutive law [13,14].

In order to assess the safety of the church's masonry structure and clearly justify the damage described in the previous section, two different models of increasing complexity have been adopted: modelling, which resorts to the finite element method, includes linear elastic and nonlinear behaviour of the material. As previously stressed, it is essential to verify the adequacy of the models with the existing building; this can be carried out using a number of techniques, namely flat-jack testing or dynamic identification (namely “in situ” testing), but also by a comparison with the damage survey (in terms of cracks, displacements, etc.). In the present paper, the adopted model validation technique was the comparison with the damage exhibited by the church. Nevertheless, it is stressed that the numerical analyses were carried out with the aim at justifying the damage exhibited by the structure and at understating its behaviour with respect to different strengthening proposals.

Two three-dimensional finite element models were constructed for the structural analysis. In a preliminary stage, only the masonry elements were considered due to the lack of effectiveness in the connection between masonry and timber elements. As a matter of fact in a preliminary step no connections are assumed to be present between the roof and the confining walls.

4. Structural analysis

Analysis of the seismic behaviour of historic masonry buildings, and in particular of churches, is a quite difficult task due to: the difficult numerical modelling of the nonlinear behaviour of masonry material, with almost no tensile strength; the incomplete experimental characterisation of the mechanical properties of the masonry structural elements; and the complexity of the geometrical configuration [15].

Refined mechanical models, which accurately predict the behaviour of masonry material and elements, have been proposed in the inherent literature [16–20,13]. Such models adopt different strategies to take into account the highly nonlinear behaviour of the material both in tension (low tensile capacity and consequent cracking phenomena) and in compression, and some of them are also able to provide the structural response to large cyclic deformations, which occur under seismic actions. Unfortunately, they are hardly applicable
Fig. 15. Vertical stresses $\sigma_y$ (0.1 MPa).

Fig. 16. Longitudinal (along nave) displacements (cm).

Fig. 17. First mode $T_1 = 0.319$ s.

Fig. 18. Second mode $T_2 = 0.306$ s.

Fig. 19. Third mode $T_3 = 0.278$ s.

Fig. 20. North side, longitudinal displacements (cm).
to the complete 3D analysis of complex structural systems, due to the great number of parameters involved in the definition and updating of the mechanical model and the large number of degrees of freedom required for structural meshing, which lead to untreatable problems.

Nevertheless a preliminary linear elastic static and dynamic analysis of the 3D structural complex provides valuable information on the global behaviour and on the interaction among the single elementary parts, which constitute the structure.

The analysis method proposed and adopted in this paper for overcoming the above difficulties is based on a three-step procedure: (a) firstly the overall structure is analysed in the linear range through a complete and refined 3D model, with the aim of characterising the static and dynamic behaviour, defining the internal force distribution among the single elementary parts and identifying the weak points of potential failure in the building; (b) secondly the overall structure is analysed in a nonlinear range making some hypotheses and trying to understand the causes of the actual cracking path. The aim of
this step is twofold: to understand the origin of the cracking and to obtain, from a numerical point of view, the actual state of conservation of the church; (c) thirdly the nonlinear model of the church previously identified is used for a simplified assessment of the seismic behaviour of the whole building.

The comparison between the strength capacity of the single 3D model of the church delivers an estimate of the seismic safety level of the building, and gives an indication of the types and locations for the required restoration interventions.

4.1. Linear static analysis

As stressed before, a preliminary linear analysis of the masonry building is done in order to obtain some basic information on the global behaviour of the building. Although the hypothesis of elastic behaviour of masonry material is not strictly correct, this preliminary step is able, in the authors' opinion, to offer some basic results concerning the internal force distribution among the single elementary parts and also to identify the weak points of potential failure in the building. A preliminary linear analysis is also significant in order to have a feedback with the following nonlinear analyses.

Static and dynamic analyses have been carried out on the 3D model of the church structural complex using the FE computer code ANSYS [21]. The masonry walls have been modelled by means of solid45 elements (eight node isoparametric linear elastic elements). The final 3D model consists of 10 620 joints and 5963 3D solid45 elements. For the masonry brick elements the hypothesis of linear elastic behaviour has been adopted, and a smeared model with homogenized properties has been used where the masonry is modelled as a isotropic continuum. The problem of using smeared models lies in the calibration of the material parameters, [22]. Values concerning the physical properties of masonry material have been established on statistical analysis of test data found in the literature [2,11]. Particularly, the mechanical properties were agreed based on existing tests of similar masonry and in situ testing; therefore conservative values are estimated from this experience. Values assumed in this research are: Young’s modulus $E$ equal to 2000 MPa, Poisson modulus $\nu$ equal to 0.25 and own weight $W$ equal to 22 kN/m$^3$. The structural elements have been analysed under constant vertical loads, deriving from its own weight and from the roof loads. Timber trusses have not been modelled and their self-weight has been applied to the model as vertical load acting on the top level of masonry wall. In the analysis of the “as is” state of the church no connections are assumed to be present between the timber roof and the confining walls due to the
weakness of it. Timber roof trusses are simply leant against the masonry wall without any effective restraint.

In this first stage, only the masonry elements were considered due to the difficulties in assessing the actual (anyway poor) connection between masonry and timber elements. Masonry walls are assumed to be well connected between them (see Fig. 14) due to the good quality of the existing connections.

The main information which can be derived from the linear analyses of the church is the interaction, and in particular the stress resultant distribution, among its structural elements in the two principal directions. Furthermore the comparison of the results obtained from the different analyses allows us to identify the effect of the dynamic characteristics of the building on the interaction among elements and on the distribution of internal forces. In Fig. 15 the results of the static analyses on the 3D church model, under constant vertical loads deriving from the own weight and from the roof loads, are reported in terms of stress resultant distribution among the structural elements. As a matter of fact the maximum value of the compression is reached by the columns that sustain the vault of the crypt where the compression is about 1 MPa (anyway admissible for the stone material). Substantially this result confirms that churches were designed by architects who were skilled in attempting very slender schemes; these structural systems, though perfectly able to resist vertical (compressive) static loads, are not always adequate to withstand horizontal forces deriving from seismic actions (or, in general, for load conditions different from the vertical one).

Maybe more interesting than the local value of the tensional results are the displacements (see Fig. 16). The maximum value of the displacements are reached close to the big arch of the transept. This is an interesting result because it offers a clarification of a component of motion to the second mechanism: the presence of the thrust of the central semidome that doesn’t find an appropriate support at the roof level generates a variegate cracking path developed on the top level of the apse.

### 4.2. Modal analysis

The periods and modal shapes of the 3D FE models are provided in Figs. 17–19.

### Table 1

Modal effective masses for transversal, longitudinal and vertical direction

<table>
<thead>
<tr>
<th>Mode n°</th>
<th>Period (s)</th>
<th>$M_{eff}$ (% X-direction)</th>
<th>$\Sigma M_{eff}$ (% X-direction)</th>
<th>$M_{eff}$ (% Y-direction)</th>
<th>$\Sigma M_{eff}$ (% Y-direction)</th>
<th>$M_{eff}$ (% Z-direction)</th>
<th>$\Sigma M_{eff}$ (% Z-direction)</th>
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<tr>
<td>1</td>
<td>0.318</td>
<td>20.0</td>
<td>20.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>0.305</td>
<td>2.79</td>
<td>22.8</td>
<td>0.00</td>
<td>0.00</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>3</td>
<td>0.273</td>
<td>0.00</td>
<td>22.8</td>
<td>0.04</td>
<td>0.04</td>
<td>12.7</td>
<td>12.7</td>
</tr>
<tr>
<td>4</td>
<td>0.225</td>
<td>5.96</td>
<td>28.8</td>
<td>0.00</td>
<td>0.04</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>5</td>
<td>0.181</td>
<td>9.52</td>
<td>38.3</td>
<td>0.00</td>
<td>0.04</td>
<td>0.31</td>
<td>13.0</td>
</tr>
<tr>
<td>6</td>
<td>0.169</td>
<td>0.18</td>
<td>38.5</td>
<td>0.00</td>
<td>0.04</td>
<td>0.38</td>
<td>13.5</td>
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<tr>
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<td>0.168</td>
<td>2.25</td>
<td>40.7</td>
<td>0.00</td>
<td>0.04</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>8</td>
<td>0.162</td>
<td>0.00</td>
<td>40.7</td>
<td>0.04</td>
<td>0.08</td>
<td>7.84</td>
<td>21.3</td>
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<tr>
<td>9</td>
<td>0.160</td>
<td>8.13</td>
<td>48.9</td>
<td>0.00</td>
<td>0.08</td>
<td>0.00</td>
<td>21.3</td>
</tr>
<tr>
<td>10</td>
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<td>0.23</td>
<td>49.1</td>
<td>0.01</td>
<td>0.09</td>
<td>32.7</td>
<td>54.0</td>
</tr>
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</table>
Table 1 reports the effective masses of the main vibration modes in the transversal, longitudinal and vertical direction. As can be argued from Figs. 17 and 18, the first two modes of the “as is” building involve translation in the weakest transversal direction of the main nave, with significant out-of-plane deformation of the orthogonal elements. The third modal shape (Fig. 19) is a translational mode in the longitudinal direction for the clerestory transept. The fourth modal shape displays torsional deformations, revealing a certain collaboration among the transversal and the longitudinal structural elements of the abbey. Next two modal shapes (not represented here) correspond to, respectively, the second torsional mode of the church (the fifth mode) and the third torsional mode (the sixth mode). The higher modal shapes are a combination of transversal vibration mode and torsional mode. The distribution of the modal shapes demonstrates that the “as is” building, though characterised by very stiff structural elements on the perimeter, displays low transversal and torsional stiffness, and significant out-of-plane deformations of the elements. Furthermore the deformed plan configuration of the structure confirms that the seismic loads acting along either longitudinal or transversal directions of the church involve remarkable out-of-plane deformations of the orthogonal structural elements.

4.3. Nonlinear static analysis

Before proceeding with the assessment of the seismic resistance of the church, in order to investigate the origin of the actual crack pattern of the abbey, and to identify the numerical model, the structural elements have been analysed under constant vertical loads, deriving from the self-weight and from the roof loads, and under increasing linear vertical displacements under the transept area.

The brittle behaviour of masonry has been modelled through an appropriate failure criterion, here defined by means of only two material parameters \( f_t \) (uniaxial tensile strength) and \( f_c \) (uniaxial compressive strength); cross section of the assumed failure surface will be defined with a cyclic symmetry about each 120° sector of the deviatoric plane. Both cracking and crushing failure modes have been accounted for. The presence of a crack at an integration point has been represented through modification of the stress–strain relations by introducing a plane of weakness in a direction normal to the crack face. Also, a shear transfer coefficient \( \beta \) has been introduced (depending on the crack status: open—\( \beta_t \)—or re-closed—\( \beta_c \)), representing a shear strength reduction factor for those subsequent loads inducing sliding (shear) across the crack face. In order to reduce the number of the parameters employed to represent the nonlinear behaviour of masonry, a Drucker–Prager perfectly plastic criterion has been employed in the model, avoiding the need for definition of a hardening rule. In this way cohesion \( c \) and angle of internal friction \( \phi \) have been assumed as the only two material parameters required to define the yield surface. The yield surface it is assumed that does not change with progressive yielding, hence there is no hardening rule and the material is elastic–perfectly plastic. The failure surface assumed is the Willam and Warnke surface [23–25]. This failure criterion has been adopted initially for concrete accounts for both cracking and crushing failure modes through a smeared model. Despite the need for five constants in order to define the criterion, in most practical cases (thereby when the hydrostatic stress is limited by \( \sqrt{3f_c} \)) the adopted failure surface has been specified by means of only two constants: \( f_t \) and \( f_c \) (respectively the uniaxial tensile and compressive strength). Table 2 reports the selected values for the model solid 65 parameters.

Fig. 20 reports the longitudinal displacements; Figs. 21 and 22 report the crushing and the cracking path obtained in the nonlinear analysis due to increasing linear vertical displacements under the transept area. By the results it’s possible to see how taking into account a soil movement it’s able to evaluate, qualitatively, the cracking path and how
Fig. 33. Comparison of the main periods.

Fig. 34. Modal effective mass (longitudinal direction).

Fig. 35. Modal effective mass (transversal direction).
the cracking path is close to the effective one: the cracking collapse reconstructs the actual behaviour of the Farneta abbey. Moreover collapse increases after a maximum base displacement of 0.32 cm: this means that a very small value could generate a sensible cracking pattern on the church. This confirms that the main cause of the damage of the transept and the apse of the abbey is due to the modifying in the hydraulic tenor of the soil occurring during the last restoration. Cracking fissures take place in the weakest wall of the church (window area). Crack pattern is present where there is a weakness of the building (along the wall with windows or section reduction).

The soil under the abbey is a clay ground particularly sensitive to change in the water level. The last restoration that has modified the hydraulic tenor of soil, with the corresponding reduction of the water level, could be considered one of the major concerns for the cracking on the transept.

5. Seismic analysis

In order to simulate the behaviour of the abbey under seismic loads the church was subjected to an equivalent static analysis through the application of horizontal forces perpendicular to one another (allowed by several national standards in many countries and by EC8 too [26]). In particular these forces, not acting simultaneously, are evaluated upon the Italian rule [27]. Adapting the static approach inertia forces acting on the walls can be approximately evaluated considering a force distribution according to the first vibration mode of the building with linear assumption.

A preliminary new modal analysis is done where the effective actual cracking path is taken into account. In order to reduce the computational effort the previously described FE model of the church has been updated posing gap elements (namely contact52 elements) around the areas where cracks are present.

Results concerning this preliminary step are reported in Figs. 23–25. Due to the crack pattern it’s possible to observe an increase on the main periods (see Fig. 26) and a reduction on the mass participation factor for each mode (see Fig. 27). Table 3 reports the periods of the main vibration modes for the “as is” model and the identified one.

The seismic analyses carried out on the identified nonlinear 3D FE model has permitted us to evaluate the ultimate strength capacity of the church (with respect to the Italian rules). Some load combinations have been carried out for all transversal and longitudinal seismic directions allowing a direct, though approximate, assessment of the seismic safety level of the

![Fig. 36. First mode \( T_1 = 0.2454 \text{s} \) for strengthening [C].](image1)

![Fig. 37. Second mode, \( T_2 = 0.1894 \text{s} \) for strengthening [C].](image2)

<table>
<thead>
<tr>
<th>Table 3</th>
<th>Comparison of the main period for “as is” building and identified one</th>
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<tbody>
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<td></td>
<td>Farneta abbey (“as is”)</td>
</tr>
<tr>
<td>Mode n°</td>
<td>Period (s)</td>
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<td>1</td>
<td>0.318</td>
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<tr>
<td>2</td>
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<tr>
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<td>0.162</td>
</tr>
<tr>
<td>9</td>
<td>0.160</td>
</tr>
<tr>
<td>10</td>
<td>0.141</td>
</tr>
</tbody>
</table>

church. Square root of sum of squares (SRSS) technique has been used for the load combination. The most severe load combination for the building turns out to be the seismic load
acting perpendicular to the along-nave direction: it is of course due to the low stiffness of the building in this direction. For this load combination the comparison demand vs. capacity (about 10% of seismic load) confirms the susceptibility of this type of building to extensive damage and possibly to collapse, as has frequently been observed during earthquakes. As a matter of fact building collapses for a seismic load equal to about 8% of external load and collapse is due to the failure of the nave of the church where first mode (out-of-plane displacement) takes place.

Figs. 28–30 show the cracking and the crushing path along the nave of the church and of the transept wall (concerning the main arch). By a comparison among the load conditions taken into account for the seismic analyses (acting in the x and z directions) it has been observed that the church is especially vulnerable in the transversal direction, due to low stiffness and the presence of cracks in the wall arranged parallel to the seismic action.

6. Strengthening restoration

The analysis of the static and seismic behaviour of the church has permitted us to point out the two main weaknesses of the building. The first one is the lack of connections between orthogonal masonry walls. The second one is the need of consolidation of the foundation area around the apse responsible of the opening crack along the transept and the apse area. With respect to this weakness it’s then possible to suggest a strengthening retrofitting technique respectful of the architectonical aspects. Concerning the repairing of the crack pattern a new foundation is scheduled. Close to the actual foundation by a grid of minipiles a new one is realized along the perimeter of the transept area in order to stabilize the ground movement. Moreover, regarding the actual crack pattern on masonry walls three different techniques are scheduled, depending on the crack size: (a) injections of aerial mortar (grout); (b) injections of aerial mortar with steel anchor ($\phi = 6–8$ mm) on both faces of a wall connected by frequent transversal steel chains; and (c) local reconstruction of the wall around the crack. The aim of these techniques is to close the cracks, but they are also able to improve the connections of the wall and to increase the tensile and shear strength and ductility [28].

With respect to the strengthening techniques, different retrofitting restorations are taken into account in order to assess their effects on the seismic reliability of the whole building. First post-tensioned horizontal steel tie-bars closely to the top level and post-tensioned vertical steel tie-bars inside the masonry walls along the entire height are foreseen. Tension on the post-tensioned horizontal steel has been determined as not to cause higher normal stress in the masonry than a tenth of the compressive strength of masonry material (which is a very frequently and simple method to design the post-tensioning force in practice). However in order to have a check on this value, four analyses have been done with different tensioned levels. Without post-tension or with a low level of post-tension along the horizontal tie there are no benefits for the seismic behaviour. A high level of tension (more than 160 MPa) could generate local damage in the area of this horizontal ties. A post-tensional level of 80 MPa has been selected. Then, concerning timber truss roof, anchorage of timber floor joists to stone masonry is scheduled. Finally, a concrete ring of beams along the sides of the building as connection walls to the roof is planned with the aim to help the structure to work as a stiff box against the horizontal seismic load.

In order to evaluate the benefits of the restoration strengthening three different models has been done for evaluating different levels of strengthening:

Model [A]: (1) Local reconstruction of cracked area of masonry walls or injection of the existing cracks with adequate mortar; realization of an effective connection between the timber roof and the longitudinal walls; (2) Post-tensioned horizontal steel chains close to the top level (see Fig. 31); (3) Anchorage of timber floor joists to a stone masonry; (4) New foundation.
Table 4

Benefits of the different levels of strengthening retrofitting

<table>
<thead>
<tr>
<th>Retrofitting restorations</th>
<th>Collapse loads (% of seismic load)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;as is&quot; building</td>
<td>8%</td>
</tr>
<tr>
<td>Strengthening [A]</td>
<td>12%</td>
</tr>
<tr>
<td>Strengthening [B]</td>
<td>15%</td>
</tr>
<tr>
<td>Strengthening [C]</td>
<td>23%</td>
</tr>
</tbody>
</table>

Model [B]: (1) Strengthening of model [A] and post-tensioned vertical steel tie-bars inside the masonry walls along the entire height in order to increase the shear and flexural strength of the masonry wall. This strengthening hypothesis has been analyzed in order to assess its efficiency. As a matter of fact before proceeding with this type of reinforcement it is important to observe the state of conservation of the masonry. The effect of the post-tensioning, if there is not a proper design, could increase considerably the stress state on the masonry anchor area producing a local crashing \[29\].

Model [C]: (1) Strengthening of model [A] and concrete ring beams along the sides of the building (see Fig. 32) \[20\].

For each of these retrofitting techniques a new preliminary modal analysis has been done, and the results (see Figs. 33–35) show a general modifying on the modal shape with a decrease of the first period and an increase on the mass participations. Particularly, with specific reference for the strengthening [C] the first four modal shapes (see Figs. 36–38) show a better cooperation with the elementary walls of the building. The presence of a concrete ring beam on the top level of the walls (connected with the timber truss roof) produces a box behaviour. Moreover, with respect to models [A] and [C], it is possible to observe that the effectiveness of the actual retrofitting techniques is reflected on the change on the failure mode. Concerning the low level of restoration [A] (that’s substantially a repairing one) the collapse mechanism is due to the failure on the nave walls: a first mechanism mode (see Figs. 39 and 40). The increase of the ultimate load is about 4%. On the contrary concerning the high level of restoration [C] the collapse mechanism is due to a crack pattern that develops on the front of the church: the second mechanism mode with typical fracture on 45° (Figs. 41 and 42). In this case, due to the presence of concrete ring beams along the sides of the building (connected with the timber truss roof) this allows the building to work as a stiff box against the horizontal seismic load with an increasing of the ultimate load of about 15%. Table 4 reports a comparison about the benefits that it’s possible to obtain in each of the strengthening techniques analysed in this paper. The most effective technique turns out to be the third. This retrofitting assures a good increase in the level of safety of the church, and it doesn’t require a severe modification of the building behaviour.

7. Concluding remarks

In order to assess the structural behaviour, and to evaluate the seismic vulnerability, of a Romanesque church the behaviour of a study case, the Farneta abbey, has been analysed under earthquake loading. For this purpose a 3D numerical model of the Farneta abbey has been constructed. A preliminary linear elastic analysis has permitted us to obtain some basic information about the structural behaviour of the building. Next
some nonlinear analyses have been made in order to assess the seismic vulnerability. A crucial task in masonry building modelling is the evaluation of the mechanical properties. In this work, concerning this step, conservative values already available from similar experience are assumed. In order to gain an understanding of the actual deterioration state of the structure the model has been identified taking into account the actual cracking path. By this procedure it has been possible to obtain a proof of the configuration of the cracks, which was found to depend primarily on the change on the hydraulic tenor of the soil, i.e. the large sensibility of this building to ground movements. Next the church is subjected to a seismic static analysis, with respect to the Italian rules, through the application of horizontal forces perpendicular to one another not acting simultaneously.

By a comparison between the stresses and the strains due to the seismic shocks acting in the horizontal x and z directions, respectively, it was observed that the church is especially vulnerable in the along-nave longitudinal x direction, due to low stiffness and the presence of cracks in the wall arranged parallel to the seismic action (the wall responsible for counteracting the seismic loads). The comparison demand (seismic loads) vs. capacity (material and topology strength) confirms the susceptibility of this type of building to extensive damage and possibly to collapse, as frequently observed.

Also, the efficiency of current techniques for repairing and strengthening have been analysed in order to evaluate their benefits. Three different levels of restoration have been investigated, and the analysis of repairing and strengthening techniques shows the effectiveness of the usual structural reinforcement in terms of increased seismic capacity. It is believed that the results and the conclusions obtained with respect to the seismic assessment of this case study could be extrapolated to the wide variety of historical Romanesque churches, and generalized for a wide masonry building category.

References