Constructive, historical and numerical analyses for seismic strengthening interventions in San Nicolò church (Catania)

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ABSTRACT: San Nicolò is the church of the huge Benedictine monastery placed in the historical centre of Catania; during its construction (begun from 1687 on a existing smaller church strongly damaged from 1669 Etna’ eruption) Catania was struck by 1693 earthquake. Later on two others seismic events lightly damaged the buildings (1818 and 1848). Now-days the Genio Civile of Catania is taking care of structural conditions of the buildings and it organised a multi-disciplinary working group to carry on historical, techniques and numerical analyses finalised to correct and consistent strengthening interventions. Punctual observations of the church process of evolution, now-days deterioration (both structural and superficial), reveals the crucial characters conditioning the seismic vulnerability. The paper presents the results of the investigations realised on the actual masonry structure, the numerical analyses carried out and finally the aseismic intervention criteria selected.

1 INTRODUCTION

In 1997, after the collapse of the Noto’s Cathedral, the Genio Civile of Catania begun to take care of San Nicolò church’s static condition and to arrange a general plan for analyses and monitoring finalised to evaluate its actual stability. The result of these investigations, which is going to be presented, will be used to realise a consistent restoration program.

Under the co-ordination of Eng. S. Cocina, the study involved experts of various specialities to carry out a multi-disciplinary work, which is characterised by an attempt to use – in a consistent way - data coming from different technical approaches.

Particularly the data carried out from the archivist analysis (Guidoboni 2001) were read in a technical point of view to underline the most important constructive phases; furthermore the constructive analysis on elements as well as the church feature was the base to specify the weaker part of the structural arrangement to be controlled by numerical analyses. Finally, the results of assessment of stability condition, was used to define the intervention criteria (Carocci et al. 2001).

2 TECHNICAL ANALYSIS OF ARCHIVISTIC DATA

Careful examination of historical data – following the punctual research carried out in different archives (Ciucchiarelli and Mariotti 2001) - permitted to clarify some significant relationship between actual state of the church and the principal events happened during its edification and later on; it can be highlight the effort made to read the original transcriptions from a technical point of view that required a preliminary work on XVIII century technical Sicilian language (Carocci 2001).
In the following we are going to refer shortly only about the main happening related to constructive feature. Before speaking about the church we can see today, it has to be said that a previous monastery existed in the same area. The ancient complex included a church – quite smaller then the actual one – which was almost destroyed by Etna eruption of 1669 (figure 1).

The historical sources tell us quite well about the damages occurred to the complex and about the monks’ decision to rebuild completely a new church bigger than the original one, and to enlarge the monastery adding a new cloister in the east side of the existing one.

First stage of church construction – begun in 1687 – involved the realization of foundations and removal of lava’s layers. Results of archivist analysis concerning the dimension of foundation excavation agree with information coming out from survey and investigation and indicate that mean foundation depth is about 6 m.

The big 1693 earthquake did not produce direct damages on the church because its elevation structures were in a very beginning stage of construction, while it strongly damaged the ancient monastery escaped to eruption destroying, but the urgency to rebuild the monastery to permit to monks to come back in their own home, meant an interruption of the church yard continued for almost 40 years.

In fact, as the available sources refers, the years from 1701 to 1730 was dedicated completely to the monastery rebuilding and only in 1730 the archivist data report payments for the “Chiesa Nuova”.

When in 1735 the monks went to stay in the new monastery also a church must be ready to permit the usual day life of the monastic congregation. So it could be thought that between 1730 and 1735 a part of the new big church was arranged and covered by roof to be used, while in the remaining parts the construction was going on.

By the interpretation of the sources it can be said that this temporary church was made in the area of today’s transept and contiguous squared chapels, without finishing the foreseen elevation height of walls as well as without realizing the covering vaulted structures.

Another stop in the church’s construction happened in 1739 and went on for 8 years; it is probably in this period that the original plan of a symmetric monastery with the church placed in the middle was abandoned (figure 2). With reference to this subject it is interesting to underline that such a plan was well known in Catania as can be understood by observation of a city map edited in 1760 (figure 3), in which the Benedictine complex is represented just with that symmetric layout as it had to be in the original constructive program.
Figure 2: Monastery maps in a hypothetical reconstruction by J.I. Hittorf - L. Zanth, and in an early XIX century’s survey.

Figure 3: Catania view by F. Orlando, 1760; the drawing reports original symmetrical plan of the monastery, in spite of at that date the church was not finished and the general layout already modified.

Construction works began again in 1747 in the central apse area and went on in the following years continuously when in October 1775 a ruinous collapse happened in the northern part of the church’s building. At that time the construction yard was in full activity: as referred in the sources, masonry wall were going to be completed in all their height and masonry arches connecting the walls their selves were going to be realised. The big effort to conclude the church is demonstrate from the huge number of workmen presence as well as the economic expenses of that period. It can surely be said that the collapse happened when both the hemispheric vaults covering the squared chapels and the barrel vault of transept did not yet build; this information derives from the observation that in the sources we found the payment for the material to be used just to realize them. Only a generic information on location of the collapsed structures in the northern part of the church is contained in the sources, while a lot of detailed information can be derived by their analytic interpretation based on the description of the following interventions carried out on the contiguous damaged structures.
Archivist data are clearer on the causes of the collapse where they refer the opinions of two experts who visited the yard after the disaster: both of them agree to ascribe the collapse to heavy construction defects in realizing the masonry walls and pillars, and particularly to their bad internal texture realised with the presence of voids and without interlocking between the stones. As also we today well know, this kind of defects cannot be recognized looking a masonry structure from its surfaces; this uncertainty on constructive quality of the remaining walls conduced to the necessity to intervene on all of them to avoid the possibility of other structural problems. So immediately the works were divided into two different types: while a part of workmen attended to eliminate the ruins produced by the collapse and to demolish the contiguous damaged parts to be reconstructed, other workmen began to improve the texture of the standing parts by introducing big squared blocks of lavic stone.

So using this historic information it was possible to recognise by observation of actual masonry surfaces the extent of the works realised after the collapse.

The worry about structural defects continued for years; in fact, in 1669 just before the dome’s construction it was decided to disassemble one of the pillar that was going to support the dome and to rebuild it with big squared blocks.

The dome’s construction took place in almost ten years; archivist sources permit to know how it was built and which materials were used. It is interesting to observe that the builders were aware about the structural behaviour of vaulted systems if they put a metallic ring at the top of the drum just before the dome’s impost, and if they used light pumice stone to realize the dome structure in order to avoid excessively foundation loads (figure 4).

In 1780, when the dome was hardly concluded, some cracks appeared in the pillars and in the big arches supporting the dome itself. The two expert reports found agree that such crack pattern was not worrying for church’s stability and describe it as related to the settling of the structure after the dome’s construction.

When the February 1818 earthquake arrived the church construction was concluded from only some decades, nevertheless some damages occurred: leaving out to tell about not structural problem we have however refer that some damages affected the northern part of the church and particularly on foundation of S. Giuseppe’s chapel which were repaired in the months following the seismic event.

Last information on seismic damages concerns 1848 earthquake, which provoked some localised damages; besides the collapse of some jutting blocks of the internal cornice, the sources indicate both the movement of a lantern’s external column and its repair, which consisted in the disposing of the metallic ring we can still see today.
3 CONSTRUCTIVE TECHNIQUE AND STRUCTURAL ASSEMBLY

All of the in situ investigations, both non-destructive and destructive, carried out on the church allow distinguishing the presence of different masonry typologies, in terms of stone unit dimensions, shapes and mechanical properties. Such differences deeply affect the seismic behaviour of the whole structure, as the damages, probably occurred just because of past earthquakes, seem to prove.

One of the most relevant features concerns the constructive quality of the pillars. In comparison with the perfectly regular texture made of accurately squared blocks of the other pillars, the pillars between the first two spans show undoubtedly a worse quality: most stone units are irregularly shaped and the numerous big sized voids among them involve the presence of an excessive mortar quantity. This implies that the transverse containment ensured by virtue of friction between regular overlapping layers is drastically reduced. Moreover, a discontinuity has been noticed between the lower (up to a level of 2.0 m on the floor) and the upper part (up to the level of the arch impost) of these pillars.

All the same interesting is the constructive technique, widely spread in south-eastern Sicily, that has been used in the vaults and chapels: small (about 15 cm) irregular pumice-stones disposed, with the aid of gypsum mortar, in a texture of concentric layers. The same kind of stone is used in the dome too: the differences lay in the greater dimension of the units, in the replacement of the gypsum with lime mortar and in the presence of brick fragments to regularize the exterior surfaces. Such a particular texture, often employed in Catania, is also used in the drum except the window-posts and the lintels built with squared calcareous stones.

As regards the structural assembly, the church shows a particular layout that can be described as a mixture between the classical basilica’s shape with three aisles and a Latin cross shape with the presence of a dome at the intersection of nave and transept (figure 5). Moreover, the hierarchy between the nave and the aisle cannot be immediately recognised because they have, atypically, the same height.

Despite such particularities, from a structural standpoint, the basilica’s layout is certainly prevailing: this means that the church exhibits a substantially different seismic behaviour in the longitudinal and in the transverse direction: this latter represents the weakest direction and, consequently, the numerical analysis can be limited to it. More precisely, two sections can be identified in the transverse direction: they account for different border conditions and refer, respectively, to the nave and to the transept.
4 MECHANICAL MODELS

For the evaluation of the seismic safety of the church of San Nicolò l’Arena two different kinds of mechanical models have been adopted, a classical finite element model, implemented in the computer code ADINA (Bathe 1985) and a simplified model denoting the analyses performed by means of the so-called Giuffrè mechanisms (Giuffrè 1993). The former extends to masonry a numerical procedure originally developed for the non-linear analysis of reinforced concrete structures that allows following the succession of material cracking by rearranging the stiffness matrix whenever the tensile strength is overcome: the collapse is reached when such a rearrangement is not any more possible, i.e. when the structure becomes kinematically indeterminate (Bathe and Ramaswamy 1979). The latter can be considered as an application of the upper bound theorem of limit analysis and defines the collapse as the loss of equilibrium consequent to the changing of the structure into a rigid body kinematic chain (Kooharian 1953) (Figure 6).

Both models account for the small, even negligible, tensile strength of masonry work, although in obviously different ways: in ADINA the tensile strength is one of the mechanical parameters by means of which the material constitutive relation is described, while in the Giuffrè mechanisms its initial value is zero along predefined fracture planes. The limited tensile strength of masonry can be regarded as the most relevant mechanical outcome of its intrinsic discontinuity. Such a property strongly affects the whole structural behaviour of historical buildings and, if in some ways it has to be properly dealt with in any mechanical analysis, in others it allows limiting the analysis itself to the structural sections that, following a preliminary investigation, can be considered as critical with regard to the seismic behaviour of the whole building.

Coherently with the above discussed structural organisation of S. Nicolò l’Arena, two partial models have been identified, both pertaining the transverse direction of the church and representative, respectively, of the nave and of the transept (Figure 7). The results obtained with the different mechanical models show an excellent agreement, as regards both the collapse mechanisms and the peak ground accelerations (PGA). Such an agreement ensures that the proposed models are reliable enough in predicting the essential features of structural behaviour; moreover, it allows, on the grounds of the performed analyses, explaining the current damage condition of the church and, as a final consequence, deciding the antiseismic interventions.
5 STRUCTURAL ANALYSIS AND CURRENT DAMAGE CONDITIONS

The maximum peak ground acceleration (PGA) the church can withstand, in its present condition, is included in the range between $0.19g$ (Giuffrè mechanisms) and $0.22g$ (ADINA). The difference is not excessive and derives from the different tensile strength values assumed in each model. When compared with the maximum expected PGAs, for which we can refer both to code provisions - Italian ($0.25g$) and European ($0.30g$) – and to Catania project results (Faccioli and Pesina 1999) – even more penalizing – the obtained accelerations undoubtedly demonstrate the need for improving, by means of proper structural interventions, the seismic strength of the church. On the other hand, the existing damages, unequivocal sign of the occurred earthquakes, determine a significant reduction of the actual seismic strength with respect to the one resulting from numerical analysis and pertaining the undamaged structure.

In spite of its complexity, the crack pattern seems to be consistent, in many situations, with the damage that expresses, in the numerical simulations, the onset of a given collapse mechanisms.

The most relevant damages of the church, so worrying that provisional interventions were executed, are undoubtedly the diagonal cracks in the pillars between the first and the second span of the nave. Even if neither ADINA nor Giuffrè mechanisms were able to exactly reproduce the actual trend of these cracks, nevertheless the numerical results suggest a possible explanation of the particular damage of the pillars, which seems to be satisfactory enough.

First of all, it must be emphasised that the given pillars are lacking in effective buttresses structures in the transverse direction unlike the pillars of the subsequent spans and, particularly, of the pillars that support the dome.

In the analysis performed with the simplified mechanisms this circumstance is assumed as a starting hypothesis and emerge from the comparison between the seismic deformed shapes of the two examined models obtained with ADINA. The horizontal top displacements are quite greater for the first couple of pillars than for the other ones and, consequently, the compressive stresses in the former exhibit a more significant increase than in the latter when the structure is subjected to a seismic action, with respect to the static load condition. The compressive stresses in the nave pillars change from $11$ (static) to $42$ kg/cm$^2$ (seismic), while in the pillars supporting the dome they change from $18$ to $24$ kg/cm$^2$. Such an increase is even more worrying accounting for the worse mechanical quality of the cracked pillars with respect to the others, as outlined above.

Even the crack pattern both in the main arch beneath the dome and in the drum could be reasonably interpreted as a consequence of a seismic action. These are precisely: (i) the cracks in the keystones of the four main arches, (ii) the numerous, even if often just appreciable, diagonal cracks in the window-posts of the drum (these are more marked in the windows corresponding to the major axes of the church), (iii) the cracks in the lintels of the drum windows (on the contrary, these are more marked for the windows corresponding to the pendentives). These cracks as a whole seem to be consistent with a horizontal relative displacement, even small, of the impost of the main arches: such a movement would in fact give rise, in succession, to (a) the lowering of the centre of the arches, with the appearance of the keystone cracks at the intrados, and, consequently, (b) to the changing of the internal thrust lines from an almost vertical diffusion (un-
cracked state) to an arch diffusion compatible with the current cracks in both the lintels and the window-posts of the drum.

The seismic deformed shape (obtained with ADINA) shows as the transverse horizontal displacement of the transept model is associated with a notable lowering of the middle portion, that is the one corresponding to the dome, further accentuated by a small relative displacement of the arch imposts: what means that the seismic action can really produce the crack pattern above described. Anyway, acting as buttresses, the transept walls prevent these movements from assuming values that could prejudice the dome stability.

The cracks that almost completely cut the base of the drum also receive a persuading explanation from numerical analyses. The results obtained from both the FEM and the simplified mechanical models point out that, at the base of the drum, the shear on normal stress ratio (or, equivalently, the horizontal on vertical component of the total load ratio) turns out to be anyhow greater than 0.5, for a PGA of 0.22g (and reaches the remarkable value of 0.68 for a PGA of 0.30g). The value of 0.5 justifies the current observable damages at the base of the drum and appears, indeed, unacceptable in the hypothesis that the shear strength, along the cracked planes, is at present exclusively due to the friction.

Even if for undamaged masonries the shear strength is determined, in addition to friction, by other important contributions, such as mortar cohesion or stone interlocking, for cracked masonries such contributions are not any more effective. In this case, the seismic strength is drastically reduced and can be easily overcome during an earthquake that involves a friction demand comparable to the aforesaid values.

As regards, eventually, the horizontal cracks along the parallel circles of the little domes in both aisles, as well the horizontal cracks of smaller width in the vault of the nave in the second span, the numerical analyses do not offer resolutive suggestions. With both models isolated cracks are obtained that can be only slightly referred to the actual ones. However, just the limited extent of the numerical crack pattern allows interpreting the current damages of both the central vault and the little aisle domes as an induced damage, consequent on the kinematic mechanisms that mainly involve other structural elements.

So, the nave vault takes part in the mechanism of the nave section the most relevant effect of which lies yet in the diagonal cracks of the pillars; equally, the little aisle domes suffer the effects of the movements of the pillars supporting the main dome with respect to which they exhibit a much smaller stiffness.

It is only just worth commenting the cracks along the meridians of the main dome. They clearly prove a classical behavioural feature of the axisymmetric domes in which the tensile stresses, unavoidably present in the parallel circles near the impost, can produce even significant damages along the meridians when the material of the dome is not strong enough in tension. The greatest problem deriving from the appearance of the meridian cracks could lay in the need for opposing to the horizontal thrusts which, in the undamaged dome, are partially supported by the tensile strength of the material. For this problem the traditional solution consists in hooping the dome.

6 DESIGN CRITERIA FOR THE RESTORATION PROJECT

The design suggestions for improving the seismic behaviour of the church and resolving the aforesaid problems assume as a general, both cultural and mechanical, criterion the material and technical consistence of the interventions with respect to the existing building. Such a criterion has been lapidarily expressed by Antonino Giuffrè when he asserted that masonry structure should be restored employing masonry technique.

As it has been told above, the most important mechanical characteristic of the masonry work is the precariousness, sometimes the total lack, of internal constraints: it follows, almost naturally, that the main objective of any structural restoration project should be, in order of gravity, the introduction of the lacking constraints, the improvement of the ineffective ones and the re-establishment of those which have been modified.

As a general antiseismic intervention, tie-rods in both transverse and longitudinal direction are proposed in S. Nicolò l’Arena. Some of these ties perform the obvious function of connecting together the external walls of the church and represent, in a certain sense, a fixed design solution; someone else are explicitly directed to prevent the seismic collapse mechanisms that emerge from
the structural analyses (such as the ties proposed for the extrados of the main arches of the dome and for the walls upon the aisle arch), both reducing the maximum foreseeable displacements of limited structural portions and restricting the cracking phenomena to which the onset of relevant kinematic mechanisms is associated.

Figure 8: intervention proposal for cracked pillars.

Figure 9: intervention details.

The intervention proposed for the two diagonally cracked pillars can be explained in terms of increase of the current constraint degree, too; indeed, this was not fully satisfactory also before the pillars were cracked and the constructive analysis above discussed clearly shows it. In order to restore their original transverse constraint, and improve it, the pillars could be hooped for the whole height or the main order (Figures 8 and 9). The hooping can be realised by means of steel small bands, so as to easily put them under the plaster (after a local removal of limited portions of the strips), provided that the lack of flexural rigidity of the thin band is counterbalanced by a sufficient number of ties in the thickness of the pillars.

The most ticklish question related to the intervention on the pillars is that of the hooping spacing: if the spacing is too loose the hooping reveals ineffective while, on the contrary, an excessive
thickening would lead to an unnecessary strengthening. The proposed solution seems to be reasonable, even if more refined numerical analyses could be conveniently performed.

Eventually, as regards the interventions for the drum, although all the technical details are not defined yet, the criterion that will be followed is quite simple. Since the problem is the substantial decrease in shear strength, as a consequence of the described damages, it is suggested to insert along the horizontal cracked planes an adequate number of stone toothing in order to prevent the possibility of a sliding failure by involving, in the shear mechanism, the uncracked portions of the pillars: these mobilize, besides friction, the cohesion of the material too and offers, consequently, a quite greater strength.

7 CONCLUSIONS

A methodology for evaluating seismic safety of historical masonry buildings and defining, when needed, proper structural interventions have been presented. Rather than discussing the specific results obtained for S. Nicolò l’Arena, it is worth underlining the relevant features of the methodology.

What mostly characterise it is, first of all, the strict interaction among historical and archivist research, in situ investigations and structural analysis and, besides, the influence of these operations as a whole on the final definition of the project.

The historical and archivist researches are indispensable for both preliminarily defining the most suitable location and number of in situ investigations and assisting the interpretation that can be given of the emerging results. They allow, jointly, achieving a thorough knowledge of the historical constructive techniques, employed in a given building, and of the mechanical quality these techniques can exhibit. Moreover, on such a basis, the most appropriate structural models can be defined. A rational choice consists in adopting different models, with different degree of accurateness and refinement, in order to reciprocally calibrate them; never disregarding, however, the capability of any model to account for the essential mechanical features of ancient masonries.

Eventually, all the acquired data and results enable an aware definition of the most respectful structural interventions: seismic safety can be so reached without losing the heritage the monument deliver us.

REFERENCES


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