Reinforcement of the 18th Century Buttresses of Maniace Castle: Design and Execution

REINFORCEMENT OF THE 18TH CENTURY BUTTRESSES OF MANIACE CASTLE: DESIGN AND EXECUTION

C.F. Carocci[(1)], C. Circo[(1)], G. Cocuzza Avellino[(2)], R. De Benedictis[(3)], N. Impollonia[(1)], C. Tocci[(4)]

[(1)]*Dipartimento di Ingegneria Civile e Architettura, Università di Catania, Viale A. Doria 6, Catania, Italy*

[(2)]*Professional Architect, Siracusa, Italy*

[(3)]*Professional Engineer, Siracusa, Italy*

[(4)] *Dipartimento di Architettura e Design, Politecnico di Torino, Italy*

REINFORCEMENT OF THE 18TH CENTURY BUTTRESSES OF MANIACE CASTLE: DESIGN AND EXECUTION

ABSTRACT: Maniace Castle, built on Ortygia island in Siracusa (Italy) in the first half of the 13th century and hosting a magnificent hypostyle hall (the *Salone*), a square area covered by 25 cross vaults supported by columns, was severely damaged by the 1693 earthquake and the explosion of the ammunition dump in 1704. The buttresses, built immediately afterwards to counteract the thrust of the 10 surviving vaults, did not guarantee adequate seismic safety so that a metal scaffolding was installed in 2001 to prevent an overturning mechanism. The paper discusses the design and execution of an external pre-stressing steel system to increase existing buttress strength and the seismic capacity of the *Salone*, starting with historical and constructional analyses and dealing with both analytical and technical aspects. The monitoring system installed to detect structural response during and after the working execution is shown, along with some of the data acquired so far.

KEYWORDS: buttress strengthening; post-tensioning; tendons; controlled intervention; seismic assessment; retrofitting; collapse mechanisms

1 INTRODUCTION

The paper gathers the results of the activities carried out on the restoration site of Maniace Castle in Siracusa as part of the research agreement between the Superintendence Cultural Heritage bureau of Siracusa and the School of Architecture at University of Catania (Impollonia et al. 2014). The main tasks involved an evaluation of the seismic capacity of the façade of the preserved portion of the great hypostyle hall (the *Salone*), which overlooks the inner courtside, and the design of an intervention to reduce seismic vulnerability whilst respecting the historical and architectural value of the building.

The current configuration of the façade is the result of major consolidation works required during the 18th century after the explosion of the ammunition dump. The tragic event tore down several vaults of the *Salone* originally covering the huge square area bordered by mighty curtain walls. The façade partially embodies two trilobate shafts – the only edge columns among those supporting the ten survivor vaults – which were buttressed immediately after the explosion by means of buttresses designed by Bourbon engineers to counteract the horizontal thrust of the ogival vaults.

In recent years, following the earthquake that struck Eastern Sicily in December 1990 and because of cracking phenomena involving the surviving columns of the hall, it was feared that the *Salone* could suffer structural problems in the case of seismic actions. The inner columns were strengthened by adding new steel hoops to the old ones, and an impressive (provisional) metal scaffolding, resting on a massive concrete foundation, was installed in 2001 to contrast the outward overturning of the trilobate columns and the adjacent inner façade.

Therefore, the design presented in this paper had to address different issues arising from multidisciplinary queries, in addition to the problem of seismic vulnerability of the façade. The problem was to increase seismic capacity of the façade whilst allowing for the safe removal of the provisional scaffolding along with its foundation, so as to restore the historical appearance of the courtyard as deriving from the traumatic event which, in eighteenth century, irreversibly altered the layout of the Castle. The proposed solution employs a vertical post tensioning steel system by means of external tendons positioned at the side of the buttresses.

The activities took advantage of an earlier structural investigation and seismic analysis conducted by Petrini & Casolo (2009) and exploited in situ testing and geometric survey previously commissioned by the Superintendence (Ipertec 2003). The valuable historical study by Bares (2011, 2016) provided thorough understanding of the damage, rehabilitation works and modifications made to the castle fabric.

Reinforcement work proceeded under the surveillance of a monitoring system installed to monitor structural response during strengthening operations and removal of the scaffolding. It will also serve for long-term inspection of the structural behaviour of the *Salone*.

2 HISTORICAL SURVEY AND MAIN ALTERATIONS

2.1 *Evolution phases*

The Maniace Castle was erected in the first half of the 13th century on the south end of Ortygia island in Siracusa by the Architect Riccardo da Lentini under the auspices of Frederick II (Bares 2011, 2016). From the outside it has the appearance of a compact square fortification with 50 m sides and 12 m in height, with four round towers on the corners (radius 4.5 m) (Figure 1, Figure 2). The strong 3.5 m thick walls at the perimeter once enclosed the *Salone*, a great hypostyle hall composed of 25 quadripartite square vaults with round arch ribs arranged into 5 bays. The vaults were supported by slender circular pillars, with the central vault resting on 4 trilobate columns.

There has been much debate about the possible absence – or different configuration – of the central vault to convey the light inside. However no definitive evidence has been found in assessing structural discontinuity (Bares 2016). On the contrary, it would seem from the rib fragments protruding from the capitals of the only two surviving trilobate columns that the central vault disclosed no such peculiarity.

The main events that modified the castle layout up to the present configuration are briefly summarized below.

The first events to affect the *Salone* were the 1542 and 1693 earthquakes. Nevertheless, it seems that damage was limited, most probably due to the partition walls among columns. These were erected just after Frederick II death, when the purpose for the great hall come to an end, so as to subdivide the large area into smaller rooms.

Some restoration work took place after the 1693 earthquake, under the supervision of the military engineer De Grunembergh. To relieve the columns, some of the vaults where unloaded, replacing the original web stones by lighter lava stones. The outcome can be seen in Figure 3. The vault infill was lightened, too (Gazzè 2001).

After a few years, in November 1704, lightning struck the castle, triggering a huge explosion in the ammunition dump. The castle was severely damaged. It underwent a series of modifications that definitively altered the original architectural arrangement.

The blast knocked down one column and five vaults (Dufour 1987) Critical damage to another three vaults led to their demolition, and six more vaults were dismantled, producing the new design that still exists today. A yard was made, bound in the south-east by the still standing vaults, two bays of the original *Salone*. On the other sides, the open space was enclosed with masonry buildings, constructed during the 18th century to replace the lateral vaults, and by the original curtain wall where the main entrance door is located.

The new layout produced an unbalanced thrust at the edge of the remaining vaults, towards the yard. For these reasons, two buttresses were positioned on the back of the trilobate columns so as to counteract the horizontal forces (Figure 4).

Further adjustments were also made in the last centuries. However they do not play a significant role in the structural flaws of the present configuration, which is mainly due to the deficient function exerted by the two buttresses, especially if seismic actions are accounted for (Figure 5).

2.2 *Constructional features*

The results of preliminary structural testing commissioned by the Superintendence (Ipertec 2003) and of the geometric survey, performed during the present research (Impollonia et al. 2014), provided useful information on the current structural flaws (Figure 6, 7). The former completed the diagnostic investigations, started in 1999 (ISMES 1999) after the internal columns had been reinforced with the still visible metal hoops, following the results of preliminary structural analyses (Modena et al. 2001; Casolo and Sanjust 2009). They concerned the whole structural system: vaults (thermography), walls and columns (ultrasonic survey, tomography, flat jack measurements), foundations (geo-radar, boring, excavations). The latter was mainly devoted to the detailed analysis of vaults and columns crack pattern and the connection between the trilobate columns and the buttresses in which they are embedded.

The internal columns supporting the remaining vaults are built up of limestone drums, whereas each of the two troubled trilobate columns overlooking the yard are given as the assembly of three monolithic marble shafts (Lazzarini 2007). Sonic surveys and tomography assessed their remarkable consistency, although some damage is evident where stress concentration is encountered.

The monolithic capitals are surmounted by limestone drums finely crafted in the *tas-de-charge* style (Bares 2011, 2016). The result is a very compact and stable support sustaining the ribs. The ribs, which are not present everywhere, are a sequence of crafted stones serving as simple supports for the webs (Cadei 1998), partially replaced by lava stones, as previously discussed. The web thickness varies between 40 and 50 cm and is sufficient to withstand the mortar and stone infill on the top, which realizes the horizontal roof.

The trilobate columns are partially embedded into masonry behind which the two buttresses were erected.

The stonework for the grouted buttresses still has an effective arrangement with regular running bonds and headers forming a coursed ashlar 50/60 cm thick with fair grout core (Ipertec 2003). The condition of the valuable masonry buttress is quite good (Figure 8) and its shallow foundation rests on high strength compact layers of yellow and white limestone.

2.3 *Crack pattern and damage survey*

The crack patterns and damage observed on the structural elements disclose the main kinematic mechanism developed before the buttresses were built (Figure 9). The contribution of the buttresses in terms of stability increment can also be assessed.

Voussoir dislocations at the pointed arches rising from the trilobate columns highlight the displacement of the vault springing under unbalanced thrust (Figure 10).

This displacement most probably occurred before the buttresses were built, produced by the 1693 earthquake or the ammunition dump explosion. The forgoing mechanism is also evident from the out-of-plumb seen on both column shafts, the average value being 6 cm (Figure 11). If a rigid rotation of the capital and the *tas-de-charge* above standing is granted, then the springer moved about 12 cm with reference to the original configuration. Such a conclusion can also be drawn from the cracks on the marble shafts of both columns, a consequence of stress concentration stemming from eccentric axial loads (Figure 12).

Although most current dislocations and displacements may be ascribed to past events (previous of buttresses construction), it must be acknowledged that the buttresses are still actively performing their function. Indeed the compressive stresses, detected by in situ experimental tests as flat jack measurements, register a maximum value at the outmost side equal to 0.73 MPa at a height of 0.96 m from the ground. Similarly, on the other buttress the value of 0.44 MPa has been recorded at a distance of 2.07 m from the ground. The outcome is clearly related to the presence of a linear bending moment produced by horizontal forces at the top of both buttresses. Then it must be concluded that a portion of the vault thrust is contrasted by the buttresses.

3 STRUCTURAL BEHAVIOUR OVERVIEW

The historical and geometrical survey along with the recognition of damage provided enough material to detect local mechanisms which are deemed to be the most critical under seismic action. They were modelled by independent 2-D macro-elements. Indeed, their structural behaviour can be satisfactorily investigated without the need for complex non-linear 3-D analyses. Rigid block mechanisms under seismic actions have been investigated by kinematic analysis, considering a 2-D model which reproduces the structural section comprising the vault, the trilobate columns and the buttresses (Figure 13).

The assumption of rigid block displacements, which supplies an upper bound of the real structural capacity, appears to be in agreement with the actual damage scenarios in Maniace Castle. Indeed, the out-of-plumb of the trilobate columns and the voussoir dislocations are compatible with monolithic stone behaviour.

This modelling strategy is explicitly envisaged in current Italian legislation (Min. Infr.&Trasp. 2008) as one of the most suitable for existing masonry buildings, for which, moreover, the adoption of refined models is often hindered by the impossibility of defining input data with the necessary precision. However, the availability of the analyses referred to in previous sections (Petrini and Casolo 2009; Casolo and Sanjust 2009) – performed on a 3-D elastic FE model (linear static) and 2-D rigid body-spring model (non-linear static and dynamic) – was exploited in order to assess the reliability of both approaches, reducing the uncertainties unavoidably pertaining to each of them. The results obtained by means of the two approaches have been compared in terms of both collapse deformed shapes of the structural system and the maximum accelerations it can withstand.

As usual, different collapse mechanisms have been explored so as to detect the one with the lowest ultimate load factor. This identifies the critical mechanism to be assumed as a reference for conducting seismic safety assessments and designing the reinforcing intervention (Figure 14 A).

It involves the outward overturning of both columns, along with the lowering of the vaults, with crack openings at the crown intrados and at haunch extrados (the latter identified with the springer of the vault over the *tas-de-charge*). The buttress rotates as much as the external column at its back. They interact through horizontal forces only, located right above the capital. No vertical mutual forces are accounted for.

It is worth noticing the peculiarity of this mechanism with respect to the classical flexural collapse of arches (Figure 14 B) (Couplet 1731, 1732), essentially related to the position of the haunch cracks, which are located (for the right side of the external span) at the intrados instead of the extrados of the vault. This position maximizes the outward rotation of the internal columns, which is consistent with the collapse deformed shape obtained by FE analyses (Petrini and Casolo 2009; Casolo and Sanjust 2009) (Figure 15) and, above all, with the damage suffered by the columns after the 1990 earthquake.

Other collapse mechanisms have been checked where hinges are located so as to produce different scenarios with crack openings at different positions on one or both vaults (Figure16). However, they are related to larger collapse load factor, as explained in detail in (Impollonia et al. 2014).

The load factor and the associated spectral acceleration, according to the critical mechanism, have been evaluated in accordance with the Italian code (Min. Infr.&Trasp. 2008). The analysis was implemented with and without the inclusion of the 18th century buttresses, in order to assess their contribution on the load factor value. The results are summarized in Table 1.

The seismic hazard related to the present configuration is not very high, as assessed by the ratio Capacity/Demand equals to 0.83. The same quantity without buttresses drops to 0.52, which proves that their beneficial effect on the structural stability is remarkable. It is interesting to note that the maximum acceleration the system can withstand in the present condition – as obtained by mechanism analysis – is substantially comparable with the value deriving from the above mentioned FE analyses (Petrini and Casolo 2009; Casolo and Sanjust 2009).

The significant capacity of the system to withstand horizontal actions, even without the buttresses, ascertains the presence of a safety margin of the *Salone* under vertical forces, albeit small, and proves the role of the trilobate columns and the contiguous and superior walls to counteract the vault thrust.

The same result can be derived following a different reasoning and evaluating the thrust of the vault by the funicular polygon corresponding to the critical collapse mechanism (Impollonia et a. 2014). This thrust (about 272 kN) gives rise to an overturning moment with respect to the outmost point of the column base, M_D, which is balanced by a stabilizing one, M_C, produced by the self-weight of the half vault and the above filling (about 1192 kN) and of the column and the perimeter wall (about 438 kN). The ratio $M_C/M_D = 1.16$ confirms the presence of a small safety margin, as also derived by kinematic analysis.

However, the amount of compressive stress registered on the buttresses by flat jack measurements (§ 2.3), is much higher than the one produced by their weight, so that an eccentric load must be present. Most probably this is the result of the additional bending moment produced by a quota of the vault thrust.

Assuming that the thrust is transmitted just above the *tas-de-charge* (at about 7.2 m above the ground), the portion counteracted by the buttresses, according to the measured compressive stress, is about 25 kN (mean value for the two buttresses), about 10% of the total thrust under static conditions.

Therefore, at present, the buttresses are mildly put in action and they can be further exploited against additional horizontal forces. Indeed, as previously discussed (Table 1), they increase the Capacity/Demand ratio from 52% to 83%.

This conclusion played a key role in the choice of a suitable strengthening design. The buttresses actually perform an important task and guarantee reasonable seismic safety. The rehabilitation strategy should acknowledge their role by eventually supporting and increasing their contribution.

4 THE STRENGTHENING DESIGN

The design strategy, therefore, was to increase the effectiveness of the 18th century buttresses in counteracting horizontal forces. This was achieved by an external post tensioning steel system to augment the existing buttress strength and stiffness by transmitting an eccentric compressive axial load. The system relies on twin vertical tendons on each side of the buttresses, anchored on the top of the buttress to a stiffened steel plate and on the bottom to a steel joint restrained on the ground by small diameter piles (Figure 15).

The top plate is designed by assembling standard structural steel elements. Namely, two UPN 140 beams and an IPE 140 beam are embedded between two stiffened steel sheets to spread the stress over the buttress head (Figure 18). The resultant of the forces transmitted by the tendons has 30 cm eccentricity (toward the vaults) to counteract the overturning moment produced by the vault thrust.

The tendons are 36-mm diameter Dywidag threaded bars. Their post-tension is set so as to maximize the seismic capacity under the following restraints: (i) limit displacement (in the opposite direction to that produced by thrust) so as to not alter significantly the actual equilibrium condition; (ii) manage thermal loads and tension drop. Finally, a post-tensioning load of 400 kN was chosen on each buttress.

Three piles are envisaged for each couple of tendons. They have 8-cm diameter and 3.6-m depth so they can be fastened to the firm soil for a length of about 2 m. The tendons have a 36 mm diameter and a collapse load of 1000 kN. The steel joint connection between the tendons and the piles is shown in Figure 19. In the execution phase, the steel joint has been dismissed, as will be detailed in § 5.1.

In order to sustain the horizontal forces at the roof level, some rods are positioned over the terrace in order to tie the inner façade overlooking the yard to the strong external wall.

The proposed intervention fulfilled the requirement to preserve the present shape and dimensions of the buttresses.

4.1 *Kinematic analysis of the design configuration*

The effectiveness of the proposed design was evaluated by means of supplemental kinematic analyses where the vertical tendons and horizontal ties have been introduced.

The added steel elements do not modify the critical rigid block mechanism, as compared to the present configuration, although the horizontal ties prevent the inner façade from rotating along with the columns, in monolithic fashion, due to the upper support (Figure 20 A).

With the aim of weighting the benefits of each intervention - vertical tendons on the buttresses and horizontal ties to sustain the inner façade - a kinematic analysis has also been carried out where the latter is not present (Figure 20 B).

The tie axial force has been limited to a maximum value of about 10 kN, corresponding to the sliding collapse of the external anchorage on the inner yard façade.

Under seismic action, the tendons will transfer a vertical load of about 820 kN, deriving in part (400 kN) from the post-tensioning load and, for the remaining part, from the elongation of the tendons due to buttress rotation (according to rigid block mechanism assumptions, this rotation corresponds to a horizontal displacement at buttress crown smaller than 4 cm).

The resulting ultimate acceleration factors are listed in Table 2, which shows the effectiveness of the proposed design, even in cases where the horizontal ties should lose efficiency.

4.2 *Non-linear analysis of the proposed design*

The results derived from the kinematic analysis, resting on rigid block mechanism assumptions, are supported by a non-linear investigation confined to the 18th century buttresses.

Auxiliary numerical analyses are also devoted to a sound evaluation of stresses and displacements on the buttresses after the proposed intervention under static and seismic actions.

The interplay of the following loads is modelled in the analyses: the dead load, the inertial forces, the pre-stress load transferred by the tendons, the horizontal force transmitted by vaults and columns.

The latter is set to 25 kN, under static conditions (see §3), and 148 kN, under seismic conditions (according to the two simplified sub-models in Figure 21 accounting for the equilibrium of the buttress and of the remaining structure).

The Takeda compression-only model is implemented to describe masonry behaviour, whose parameters are tuned according to *in situ* investigations (Ipertec 2003) and previous numerical analyses (Petrini & Casolo 2009): mass density ρ=2500 kg/m^3, elasticity modulus E=8000 MPa, Poisson coefficient ν=0.2, ultimate compression stress σ_R=8 MPa, ultimate tangential stress τ_R=0.15 MPa.

A shell 2-D finite element discretization is adopted for the masonry buttress, whereas truss or beam elements are utilized for the steel elements.

Three different scenarios are addressed: (i) present configuration (buttress without any reinforcement) under static settings (horizontal force at column capital level set to 25 kN); (ii) design configuration under static settings (axial compression on the buttress set to 400 kN applied by external tendons) and including thermal effects; (iii) as the previous setting with the addition of the seismic action seized as an horizontal force of 148 kN at column capital.

The results can be summarized as follows.

(i) According to the non-linear model, the stresses detected by *in situ* measurements are consistent with a horizontal force at capital height equal to 25 kN (consistent with the value determined according to rigid block analysis, see §3). The corresponding displacement at buttress head is about 3 mm outward and 50% of the base section is reacting (Figure 22).

(ii) The axial compression is induced by a given displacement imposed at tendon bottom end. The desired compressive action is achieved, after tension drop takes place, by applying an initial total tension of 512 kN (sum of the four tendons).

The consequent displacement at the top of the buttress is less than 3 mm inward and the entire base is now reacting with a maximum compressive stress lower than 0.5 MPa (Figure 23). The stress variation caused by thermal loads (± 25°) is estimated at around 10%, assuming a thermal coefficient for masonry and steel of $9E^{-6}$ °C^{-1} and $12E^{-6}$ °C^{-1}, respectively. This perturbation does not alter appreciably the final safety evaluation, as its effect is further reduced during seismic action, when the tendon force increases.

(iii) Finally, when seismic forces are accounted for, the displacement at buttress head is less than 60 mm outward, whereas at the lower column capital height it is about 50 mm (Figure 24). Stress concentration over 6 Mpa is confined between the first 50 cm of the buttress basement, where a 40-cm deep portion is reacting. At the same time, the maximum axial force on a single tendon is about 235 kN, and on a single pile 156 kN. These are the design loads considered for the structural element verifications.

5 THE EXECUTION PHASE

5.1 *The data acquisition system for monitoring of structural behaviour*

The monitoring system installed in the castle before the scaffolding was assembled in 2001, mainly to monitor movement across surface cracks along mortar joints and inclination of the central shafts of the *Salone*, was upgraded with new recording equipment before the intervention on the buttresses took place. The aim was to (*i*) monitor the structural response during strengthening operations and removal of the scaffolding; (*ii*) perform long term data acquisition, needed to track crack breathing; (iii) monitor axial force on the tendons and the out-of-plumb at the trilobate columns partially embedded into the buttresses to be strengthened.

The general layout of the instruments is shown in Figure 25. It comprises:

a) 11 crack meters at existing cracks (Cm1-Cm5, Cm8), at the voussoir couples of the pointed arches bounding the dislocations (Cm6, Cm7 and Cm9), like the one in Figure10; at the joint between the trilobate column and surrounding masonry behind the buttress (Cm10 and Cm11);
b) 6 biaxial high-sensitive tilt meters mounted on the capital of the shafts to measure column inclination;
c) 1 temperature-gauge and humidity meter to determine the degree of correlation between castle movements and temperature/humidity;
d) 10 shafts anchored on top of the capitals to host laser distance meters so as to detect relative displacements between the head of the columns;
e) 4 electrical resistance load cells to measure the axial force on the tendons.

5.2 *On site working*

The four horizontal tie rods, sustaining the inner façade with stiffened anchor plate (Figure 26), were first laid on a narrow trench on the terrace.

Main on site work devoted to buttress strengthening started by positioning the two stiffened plates on top of each buttress after levelling their upper surface with mortar and a copper coating (Figure 27). Each plate weighs 4.3 kN and was raised by ropes on an inclined plane fixed to the existing scaffolding (Figure 28), which was to be dismounted only after retrofitting completion.

The drilling of the piles took place immediately after. In a departure from the design plans (Figure 19), only two piles were executed on each side of the buttresses, aligned with post tensioning bars. The new solution did away with the steel joints and was adopted mainly for aesthetical reasons. At the same time, the drilling operations were simplified. On the other hand, small rotations between the tendons and the piles, which would have been allowed by the steel joint, are not possible, so that the steel bars may undergo bending moment. The reduction in the number of piles involved an increase of their diameter and anchorage length, respectively augmented to 10 cm and 5 m. The same steel threadbar diameter and model was adopted for the tendons and the anchors which are coupled by a static coupler as shown in Figure 29.

The post-tensioning of the bars was carried out in three steps over a period of 15 days, to check the short time effects of creep at mortar joints and process the data of the monitoring system. In the first step, a tension of 40 kN was imposed on each tendon. After two days, a supplemental torque was exerted so that the axial force was set to 60 kN. At the third step, the axial load reached the value of 75 kN. Although the design value is 100 kN, it was decided to stop at a lower force in order to monitor castle displacements and rotation along a larger time window before imparting the design post-tensioning load.

A few days after the third step, the scaffolding was removed starting from the central portion and advancing towards the sides behind the buttresses (Figure 30). It must be mentioned that most of the contact plates between the masonry blocks and the temporary metal structures where just loosely tensioned, proving that the opposing force exerted by the scaffolding was quite limited. Indeed, the scaffolding was manly erected to withstand horizontal seismic forces, as the buttresses were adequate to counteract static vault thrusts. After tubes and couplers were disassembled, the concrete foundation reinforced by steel beams was demolished so that the inner façade was free of any obstruction (Figure 31**Errore. L'origine riferimento non è stata trovata.**)

No tension loss has been recorded so far on the monitored tendons, although it will almost certainly arise later due to long term creep. The small thickness of buttress mortar joints should guarantee moderate tension loss, which could be promptly counteracted by a supplemental use of the torque wrench.

The axial force at the tendons is moderately affected by temperature variation, the load cell registered an axial force variation of approximately 0.6% per °C, which is 50% higher than what had been predicted analytically.

No movements which could be ascribed to the three-steps tensioning operation and disassembling of the scaffolding have been recorded by the monitoring systems during yard activities. Most measure variability recorded at sensors a) and b) is caused by temperature variation. At some tilt meters the correlation with temperature is very high as is the sensitivity to its variation, as evidenced in Figure 31. The figure plots the temperature registered at the work site during the days the main reinforcement procedures were carried out, as also the simultaneous inclination acquired by Tm1 and Tm6 along y-direction (see Figure 25). It is worth noting that thermal expansion and contraction determine outward and inward rotations, respectively, for both trilobate and inner columns even though much smaller for the latter.

The results are consistent with the mechanical behaviour of the system (as emerging from kinematic mechanisms under static and seismic loads), because the column capitals give way both for vault thrust and vault expansion, with greater amplitude towards the courtside. Furthermore, the results highlight, as would appear obvious, a greater sensitivity of external columns to temperature changes.

The data recorded so far do not evidence any unexpected behaviour following the reinforcement works, which did not appreciably alter the existing stress conditions.

6 CONCLUSIONS

The paper illustrated the main design strategies and technical solutions proposed for the strengthening of the 18th century buttresses supporting the inner façade of the *Salone* in Maniace Castle, the main task being to reduce its seismic vulnerability whilst respecting its historical and architectural value. The execution of the intervention is also detailed, accounting for the precaution taken to monitor eventual unexpected structural response during on-site operation. The monitoring system is also aimed at long term monitoring of the structural behaviour so as to detect tension loss at post-tensioning bars.

If we were to sum up the main features of the proposed strengthening solution, we could refer to three different aspects.

First of all, the intervention was conceived to allow for a sequential dismounting of the provisional metal scaffolding without requiring further provisional equipment during the construction stage. The insertion of the post tensioning system requireed some work just at the buttresses sides and the crown so that most of the metal scaffolding was removed on completion of the reinforcement.

Secondly the design did not appreciably alter the present equilibrium state and stress patterns, as confirmed by real time data acquisition and processing. The design solution exploited the existing buttresses by optimally taking advantage of the masonry elements with the help of a post tensioning system. This involved a redistribution of the stresses over a larger portion of the buttresses and a reduction of peak values. No displacements towards the *Salone* were recorded during bar post-tensioning. Taking into account the small value predicted by calculations (less than 3 mm inward), this is probably due to small plastic deformations of the rubble masonry in which the trilobate columns are embedded (and the buttresses lie against).

Finally, the strengthening solution involved only limited alterations to the original masonry works and could eventually be easily removed in the future, being almost completely reversible.

7 REFERENCES

Bares, M.M. 2011. *Il Castello Maniace di Siracusa*. Siracusa: Romeo.

Bares, M.M. 2016. *Reconstruction of the rib vaults of the hypostyle hall in the Maniace castle of Siracusa after the 1693 earthquake*, International Journal of Architectural Heritage, 10:8, 101-1109.

Cadei, A. 1998. I castelli federiciani, concezione architettonica e realizzazione tecnica. In P. Toubert & A. Paravicini Bagliani (eds.), *Federico II e la Sicilia*. Palermo.

Casolo, S., Milani, G., Sanjust, C.A., Taliercio A., *Maniace Castle in Syracuse, Italy: Comparison Between Present Structural Situation and Hypothetical Original Configuration by Means of Full 3D FE Models*, The Open Civil Engineering Journal, 2012, 6, 173-187

Casolo, S., Sanjust C.A. 2009. *Seismic analysis and strengthening design of a masonry monument by a rigid body spring model: The ``Maniace Castle'' of Syracuse*, Engineering Structures 31, 1447-1459.

Couplet, C.A. 1731. De la poussée des voûtes, in *Mémoires de l'Académie Royale des Sciences*, Paris, 1731. pp. 79-117

Couplet, C.A. 1732. Seconde partie de l'examen de la poussée des voûtes, in *Mémoires de l'Académie Royale des Sciences*, Paris, 1732. pp. 117-141.

Dufour, L. 1987. *Siracusa, città e fortificazioni*. Palermo.

Gazzè, L. 2001. Documenti per lo studio delle fortificazioni a Siracusa. *Archivio Storico Siracusano.* s. III (XIV). Siracusa.

Impollonia, N., Carocci, C.F., Tocci, C., De Benedictis, R., Circo, C., Avellino Cocuzza, G. 2014. *La valutazione del comportamento strutturale della "Sala Ipostila" del castello Maniace di Siracusa in relazione alla esecuzione degli interventi di miglioramento previsti per le campate prospettanti la corte interna*.

Ipertec s.r.l. 2003. *Indagini strutturali sulle murature del Castello Maniace - Siracusa. Relazione Tecnica*.

ISMES 1999. *Castello Maniace – Siracusa, indagini diagnostiche sulle strutture murarie*.

Lazzarini. L. 2007. I marmi e le pietre romane d'importazione e il loro riuso a Siracusa. *Marmora* (3):107-131.

Min. Infr. & Trasp. 2008. Decreto del Ministero delle Infrastrutture e dei Trasporti del 14/01/2008. *Norme Tecniche per le Costruzioni*.

Modena, C., Franchetti, P., Zonta D., Menga, R., Pizzigalli, E., Ravasio, F., Muti, M., Meloni, R., Bordone, G. 2001. *Static and dynamic analyses of "Maniace Castle" in Siracusa-Sicily*, Structural Analysis of Historical Constructions, Lourenço, P.B., Roca, P. (eds), Guimarães, 933-941.

Petrini, V., Casolo, S. 2009. *Note sulla valutazione dell'ipotesi di intervento di miglioramento sismico del Castel Maniace. Relazione di consulenza allegata al progetto dei "Lavori di restauro e di miglioramento sismico della sala ipostila del castel Maniace di Siracusa"*.

Figure captions

Figure 1 Overview of Maniace Castle on the tip of Ortygia island.

Figure 2 Plan of the great hypostyle hall of the Castle (Salone) in a 1640 drawing by F. Negro (Bares 2011).

Figure 3 Restoration works on the cross vaults of the *Salone* carried out after the 1693 earthquake, involving the replacement of the original web stones by lighter lava stones.

Figure 4 Plan of a project proposal after the explosion of the ammunition dump in 1704 (Bares 2011); the Castle layout seems comparable to one at present, with differences revealing further adjustments introduced in the following century.

Figure 5 Comparison between the original layout (left) of the *Salone* according to G. Agnello (Bares 2011) and its present configuration (right) in the general survey by the Superintendence of Siracusa.

Figure 6 Plan and elevation of the façade overlooking the inner courtyard of the Castle with the two buttresses built in the XVIII century to counteract the thrust of the surviving vaults of the hypostyle hall springing from the trilobate columns.

Figure 7 Survey of the right buttress. Both buttresses present a stonework arrangement with regular running bonds and headers forming a coursed ashlar with adequate grout core. They are juxtaposed with rougher masonry work in which the trilobate columns of the hall are partially embedded.

Figure 8 A lateral view of the stonework arrangement of the right buttress. From the top of the buttresses, the surviving *tas-de-charge* portions of the vaults, which collapsed or were demolished in the 18th century, can be seen.

Figure 9 The crack pattern surveyed on the vaults' intrados is compatible with an outward movement of the courtyard façade due to the unbalanced thrust of the surviving vaults.

Figure 10 The classical sliding mechanism (as a consequence of the springing displacement) surveyed on the *voussoirs* of the pointed arches rising from the trilobate columns.

Figure 11 The out-of-plumb registered for both trilobate columns.

Figure 12 Cracks in the marble shafts of the trilobate columns deriving from the stress concentration due to eccentric axial loads.

Figure 13 Structural section comprising vaults, trilobate columns and buttresses (A), and corresponding 2-D model with rigid body discretization and mass centres' position (B).

Figure 14 Critical kinematic mechanism detected in the analysis (A). The mechanism involves an intrados crack opening at the haunch of the left vault – unlike the classical flexural mechanism (B), which is nonetheless kinematically consistent – because it entails a larger outward rotation for the inner column and, consequently, a lower ultimate load factor.

Figure 15 Collapse deformed shape obtained by FE analyses (Casolo and Sanjust 2009)

Figure 16 Different kinematic mechanisms checked in the analyses with hinges located so as to produce different crack openings.

Figure 17 Elevation and transverse section of the proposed design solution with the vertical steel tendons on the sides of each buttress and the upper tie-rods anchoring the courtyard façade.

Figure 18 Exploded axonometric view of the upper anchorage system on the top of buttress

Figure 19 Steel joint hinge connection between tendons and piles.

Figure 20 Kinematic analysis of the design solution: with both vertical tendons and horizontal tie-rods (A) and with vertical tendons only (B).

Table captions

Table 1 Ultimate acceleration factor with and without buttresses (first column) and its ratio with respect to acceleration demand at the site (second column).

Table 2. Ultimate acceleration factor of the proposed design configuration with and without horizontal ties (first column) and its ratio with respect to acceleration demand at the site (second column).