ARCHITECTURAL AND STRUCTURAL REGENERATION OF THE FORTRESS OF ARNARA-ITALY

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ABSTRACT
Architectural heritage can be rediscovered also by finding a new use for it, pursuing an appropriate regeneration that accounts for the cultural features of the territory and is at the same time respectful of the past history of the buildings. The fortress of Arnara (Latium, Italy) was chosen as a case study, since it is a perfect example of a Middle Age fortress, well preserved during the centuries despite the high seismic hazard of that area. The overall project here described tries to balance both architectural and structural aspects, where the structural intervention makes use of a tenso-structure that, at the same time, highlights some valuable architectural features of the fortress while improving its seismic resistance.

Keywords: Masonry fortress, regeneration, integration between new and existing constructions, kinematic limit analysis, seismic strengthening, tenso-structure

INTRODUCTION
The regeneration of the architectural form of the fortress has been the objective of this work, mainly driven by the need of allowing the local population to reuse it in full safety as a gathering place, for activities involving the whole community. The fortress in its current state, in fact, cannot be accessed, since past earthquakes had severely damaged significant portions of its bearing structures and the remains have been subsequently abandoned to an inexorable degradation. A possible solution was found in a tenso-structure that covers the internal courtyard of the fortress, thus recalling the presence of an ancient roof. The steel wire ropes of the tenso-structure then play a twofold role: that of sustaining the roof structure and that of strengthening elements of the existing walls, highly vulnerable to overturning. The local out-of-plane mechanisms were assessed by using linear kinematic analysis and the necessary strengthening measures were designed in order to prevent overturning. The wire ropes were designed through a purposely developed simplified method.

HISTORICAL PHASES
The town of Arnara belongs to a territory that was heavily struck during the last century mainly due to World War II bombings and that, from historical records, is affected by a moderate-high seismicity.

The fortress represents a category widespread during the Early Middle Age period: it is located in a central part of the defense structure, and it was used as a baronial residence having an enclosure and buildings around the emerging tower. Moreover, it is connected to a larger defense structure and probably its original configuration showed a wood fence around the tower itself (Fig. 1).
The castle dates back to around the end of the 8th century and it was built by the Lombard. The surrounding defense structure, instead, belong to different periods, the first of whose is supposed to date back to the 11th century. It is pretty reasonable to recognize that the first configuration of the defense walls was that shown in the first phase of the Figure 2: in fact, it is proved by a squared sandstone basement, just near the St. Nicola church.

At that time also the structural system of the fortress was involved in some changes (Figure 3), mainly regarding the position of the ancient tower, which was moved from the south to the north, to better control the main access to the village, known today as St. Sebastian’s Gate.

![Fig. 1 - View of the fortress from south side](image)

<table>
<thead>
<tr>
<th>EXPANSION PHASES OF HISTORIC FORTIFIED VILLAGE OF ARNARA</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1st phase: XI century</strong></td>
</tr>
<tr>
<td>The entrance was to the north of the village and through the door today known as Porta San Sebastiano.</td>
</tr>
</tbody>
</table>

![Fig. 2 - Main phases of expansion of Arnara fortified village; letter A denotes a squared sandstone basement on the left side of St. Nicola church.](image)

It is possible to recognize other construction phases of the defensive structure, due to a progressive expansion of the village. The first one dates back to the end of the 15th century. Finally, the last one is supposed to date back to around at 16th century, when the village expansion took place outside the defensive walls (Figure 2).
The fortress shows a high vulnerability and this is due to additions built over the centuries with no connections with the existing structures and anthropic events that involved the fortress itself. In fact, the bombing of World War II and the latest abandonment, accelerated its degradation and the weakening of the bearing walls.

The historical chronicles refer of several collapses of the Northern Tower, the first one in the mid-12th century, a short period after being built; the last one, instead, dates back to the 17th century and it was due to the earthquake that occurred in the Campanian Appennines. The data took from the macroseismic catalog show that only the intensity of the second one could have significantly damaged the mentioned tower (VIII-IX MCS). After this seismic event the mentioned collapsed walls were rebuilt, using the same laying of the surrounding perimeter walls. It is important to notice that the rigid block behavior is ensured by some transverse elements that link their external facings.

Moreover, during the 12th century, some walls were built at South-East without connections with the structures of the existing barn, as highlighted from an on-site survey (Figure 4). Besides, the surrounding walls were increased in height but it is pretty reasonable not to
consider a further vulnerability of these parts, because their construction phases are closely linked among themselves.

The material used for the internal and external walls are tufa blocks and pozolana layers for the mortar joints that have (both) the characteristic reddish color. The thicknesses of the walls are different according to the phase of construction.

Finally, it is important to point out that the cracks on the southern façade of the fortress and the presence of just fragments of battlements, are due to bombing of the village during World War II.

Fig. 4 - Absence of connections among the walls, observed during the survey; in the key-plan, point of view of the pictures.

METHODOLOGY OF INTERVENTION

The first step of the work presented here below, consisted to assess the seismic vulnerability of the existing parts of the fortress. Useful for this step was the deep study of the cracks framework, carried out thank to a visual and metric survey, taking into account several previous researches performed on the fortress (Curuni et al. 1975, Salvatori et al. 1978, Pala et al. 1998). The analysis of materials and connections among the walls, added further information that led to a definition of a reliable analytical model, used to predict possible local collapse mechanisms. In fact, according to the authors, the building is less prone to global collapse mechanisms, while mainly the absence of connections could have led to the overturning of the walls.

Thus, the new architectural elements, integrated with the existing structures, were designed to achieve both the structural safety of the structures, subjected to seismic actions, and a reconfiguration of internal spaces, to make them able to host the activities of the local community. Moreover, aesthetics aspects were taken into account, also considering the purpose of recalling some missing parts. Once performed an accurate assessment of the
seismic risk, the hazard of the site was defined, singling out the most vulnerable elements to design the strengthening measures.

The existing cracks were fundamental to define and assess the mechanisms that are already baited, using the limit analysis, according to the kinematic approach. A comprehensive intervention was achieved, taking into account the modern needs of the community and the requirements with respect to the architectural and structural aspects, conceiving the project without losing the historical atmosphere of the site.

PROJECT PROPOSALS

The main project proposals (Figure 5) regard the cover of the internal courtyard, to recall the original one proved by the presence of traces of shelf-support beams, and the restoration of the initial image of the tower, that was about twice in height before several collapses, as shown by a picture of the 17th century.

The project of the tower is in line with the main criteria of restoration: it consists in an independent steel structure, conceived to be easily removable and recognizable, respecting the ancient structures (Figure 6). This approach led to define low-impact interventions, but effective, according to the target.

The restoration of the walkway is involved in a larger aim, consisting of the creation of a panoramic point on the area that surrounds Frosinone. The access to this overhead path is ensured by the structure inside the tower: the elevator and the stairs that surround it, allow to reach the walkway level and a higher point under the cover, realized with a brise-soleil system provided by photovoltaic film between the glass sheets. The walkway is supported by means of inclined steel beams, housed in the original scaffolding holes. This way, exploiting what already exist, a close link among new and existing structures was achieved, previously reinforcing the latter ones with steel plates (Fig. 7).
Fig. 6 - The independent steel structure within the tower.
Although the above project proposals above contribute to the regeneration of the fortress, the main sign that characterized the whole intervention is the glass cover of the internal courtyard. It is supported by means of steel wire ropes, that have different roles, according to the part where are placed on. Designing its shape, a predetermined arc configuration was obtained, taking into account both the aesthetic impact and the structural interaction with the existing parts. Moreover, the least invasive measures were employed to design the supports of the new loads pattern. The steel wire ropes were conceived to link the new and existing structures, becoming a conceptual connection between the northern panoramic front and the internal courtyard. For the first one, in fact, it is a green pergola, for the latter a structural system that sustain the glass cover. Furthermore, the top steel wire is anchored to the new hypogeal auditorium, that takes place in the whole northern part of the site: the access is reachable through a catwalk system, starting from the lower level of St. Sebastian square, or from the higher level of Castel square (Figure 8).

Finally, The wire ropes become a stiffening system for the existing walls that surround the glass cover of the internal courtyard: at floors level they are anchored by means of steel profiles and rings, point-to-point; this way distribution of tensile forces of the tenso-structure is ensured, and the structure behaves like a box (Fig. 9).

**ASSESSMENT OF THE EXISTING STRUCTURES**

The existing structural elements of the fortress were subjected to verifications for the assessment of the local out-of-plane mechanisms, following the indications of the Italian Code (NTC 2008).
Limit analysis relating to the activation of any possible kinematic chain was used, considering a linear field by applying a suitable behavior factor. According to this approach, the horizontal multiplier $\alpha_0$ was computed. The inequality that has to be satisfied to ensure the safety of the structures is the following:

$$\frac{\alpha_0 \sum_{i=1}^{n+m} P_i}{M^* \text{CF}} \geq \frac{S_x(T_i) \cdot \psi(Z) \cdot \gamma}{q}$$  \hspace{1cm} (1)$$

where, on the left, there is the capacity acceleration that activates the mechanism while, on the right, the demand acceleration at the Limit State of Collapse (LSC) is computed.

Fig. 8 - The catwalk system seen from Piagge street.
Fig. 9 - The strengthening system at the intermediate floors and wood cover level: horizontal section of floor.
The quantities in equation (1) are as follows:

\( n \) number of weight forces applied to the various blocks of the kinematic chain;

\( m \) number of forces not directly imposed on the blocks, whose masses, under seismic action, generate horizontal forces on the elements of the kinematic chain, being not effectively transferred to other parts of the building;

\( \text{CF} \) confidence factor pertaining to the attained knowledge level;

\( M^* \) participating mass: \( M^* = \left( \sum_{i=1}^{n+m} P_i \delta_{x,i} \right)^2 \), computed by considering the virtual displacements \( \delta_{x,i} \) of the loads \( P_i \) application points;

\( e^* \) fraction of participating mass: \( e^* = \frac{gM^*}{\sum_{i=1}^{n+m} P_i} \);

\( S_c(T_i) \) LSC elastic spectral ordinate at period \( T_i \);

\( T_i \) first period of vibration of the whole structure in the considered direction (set to 0 if \( Z=0 \)). According to the Italian Code (NTC 2008), it can be estimated as: \( T_i = C_1 \cdot H^{3/4} \), where \( H \) is the building height from the foundation level, with \( C_1 = 0.050 \) for masonry buildings;

\( \psi(Z) = \frac{Z}{H} \) first mode of vibration in the considered direction, where \( Z \) is the height of the centroid of the linear hinge between the blocks affected by the mechanism, and \( H \) is the building height, both taken from the foundation level;

\( \gamma = \frac{3N}{(2N + 1)} \) modal participation factor (where \( N=2 \) and \( N=0 \) if \( Z=0 \));

\( q \) behavior factor, set equal to 2 (NTC 2008).
RESULTS OF ASSESSMENT

The outcomes of the verifications are shown in Table 1.

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>$a_0$</th>
<th>$M^*$ [kN/g]</th>
<th>$e^*$ (dimensionless)</th>
<th>$a_{0^*}$ [m/s²]</th>
<th>$a_{0^*}$ [m/s²]</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Northern tower</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Internal walls</td>
<td>0,284</td>
<td>82,71</td>
<td>1,00</td>
<td>3,76</td>
<td>0,97</td>
<td>✓</td>
</tr>
<tr>
<td>External walls</td>
<td>0,415</td>
<td>163,57</td>
<td>1,00</td>
<td>5,49</td>
<td>0,97</td>
<td>✓</td>
</tr>
<tr>
<td>Wall towards inner courtyard (Z=0)</td>
<td>0,051</td>
<td>31,71</td>
<td>0,93</td>
<td>0,73</td>
<td>0,97</td>
<td>✓</td>
</tr>
<tr>
<td>Wall towards inner courtyard (Z&gt;0)</td>
<td>0,124</td>
<td>14,68</td>
<td>0,89</td>
<td>1,84</td>
<td>1,54</td>
<td>✓</td>
</tr>
<tr>
<td>Wall towards cloister (Z=0)</td>
<td>0,037</td>
<td>36,88</td>
<td>0,93</td>
<td>0,53</td>
<td>0,97</td>
<td>☒</td>
</tr>
<tr>
<td>Wall towards cloister (Z&gt;0)</td>
<td>0,069</td>
<td>20,93</td>
<td>0,93</td>
<td>0,99</td>
<td>1,23</td>
<td>☒</td>
</tr>
<tr>
<td><strong>North-eastern room</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall towards inner courtyard (Z=0)</td>
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</tr>
</tbody>
</table>

The verifications were performed on the local mechanisms inferred from the cracks framework. Regarding the northern tower, the overturning of rigid portion of walls was assessed, mainly considering the part collapsed during the centuries (Fig. 10, left). It is pretty reasonable to exclude its activation, seen the results above. The vulnerability of the current north-eastern room was, instead, confirmed (Fig. 10, right), therefore a strengthening intervention is needed. At last, it is pretty reasonable to assume that the presence of fragments of battlements is essentially due to bombing occurred during World War II.

Fig. 10 - The model used for the verifications: (left) the overturning mechanism hypothesized for the northern tower; (right) the vulnerability of the north-eastern room.
THE STRUCTURAL INTERVENTION

The tenso-structure, designed to support the glass roof of the inner courtyard, is the hallmark of the whole project. It was conceived to have different roles, mainly to link the different parts of the fortress both from the architectural and the structural points of view. It gives a box-like behavior to the existing structures and distribute among them the tensile forces of the cover wire ropes, becoming a bracing system within the north-eastern room (Figure 9).

The wire ropes anchorages (designed once the tenso-structure was exhaustively assessed) at the level of the auditorium slab, were needed mainly for structural reasons. Actually, the northern wall was unable to resist to any anchorage force. The wire ropes are designed, in fact, to pass through the wall without friction (therefore without transferring pulling force), this way stabilizing it through the resultant contact forces (see force R in Figure 11).

![Diagram of the tenso-structure integrated with the existing structures.](image)

Fig. 11 - The tenso-structure integrated with the existing structures.
The work focused on the development of a recursive formula to determine the configuration of the tenso-structure. It belongs to a simplified approach according to which is possible to find the deformed shape of the wires, using the variation of the angles \( \theta_i \) between each segment of them. The steel wire ropes are considered pre-stressed by the stressing forces \( T_{\text{sup}0} \) and \( T_{\text{inf}0} \). They have different area, \( A_{\text{sup}} \) and \( A_{\text{inf}} \), and are placed at a relative vertical distance \( y_0 \). The following formulas are applicable for both top and bottom rope, by replacing the relevant quantities. Once assigned the pre-stressing forces, starting from the first angle, it is possible to determine the one acting along the first wire segment. Then, applying equilibrium at each subsequent joint, the value of the corresponding angle can be determined (Figure 12).

![Diagram of the tenso-structure](image)

**Fig. 12 - The procedure followed to determine the tenso-structure deformed shape (upper wire)**

The vectors \( P \) define the intensity of the applied loads while \( x \) is used to represent their position. The horizontal wires have length \( L \), area \( A \), and elastic modulus \( E \). Once defined the quantities, the reactions at the ends of the rope are immediately computed:

\[
R_n = \frac{1}{L} P^T x \tag{1}
\]

\[
R_0 = 1^T P - R_n \tag{2}
\]

where 1 is a vector consisting of unit elements.

By writing the equilibrium equation for each segment of the steel wire rope, it is possible to write for the generic one:
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\[ T_j = \frac{T_0}{\cos \theta_j} \]  

(3)

being \( \theta_j \) the first angle considered, and find this way, each subsequent angle considering the previous one as follows:

\[ \tan \theta_j = \tan \theta_{j-1} \frac{P_j}{T_0} \]  

(4)

The displacement of the loads application points is due to the weight of the cover considering also any variable loads, and taking into account the stretch of the vertical connection hangers.

The resulting horizontal and vertical displacements are therefore evaluated with the following simple equations, that define the final system configuration of both top and the bottom wire:

\[ u_j = u_{j-1} + L_{j-1,j} \left( \cos \theta_{j-1} + \frac{T_0}{EA} \right) \]  

(5)

\[ v_j = v_{j-1} + L_{j-1,j} \left( \cos \theta_{j-1} + \frac{T_0}{EA} \tan \theta_{j-1} \right) \]  

(6)

where:

- \( u_j \) horizontal coordinate of the point being considered along the rope;
- \( u_{j-1} \) horizontal coordinate of the point before the one being considered;
- \( v_j \) vertical coordinate of the point being considered along the rope;
- \( v_{j-1} \) vertical coordinate of the point before the one being considered;
- \( L_{j-1,j} \) length of the rope segment being considered;
- \( \theta_{j-1} \) entry angle of the segment being considered;
- \( T_0 \) pre-stressing force applied at the ends of the rope;
- \( EA \) axial stiffness of the rope.

The design of the tenso-structure is completed by the design of all vertical hangers connecting the two ropes, considering their lengths as follow:

\[ L_{\text{hanger,}j} = \sqrt{\left( v_{\text{top,}j} - v_{\text{bot,}j} \right)^2 + \left( u_{\text{top,}j} - u_{\text{bot,}j} \right)^2} \]  

(7)
The horizontal and vertical coordinates are obtained from the solution of the two wires (5)(6). The final length of each hanger is then fixed by twisting the hangers’ turnbuckles.

CONCLUSIONS

The work described above shows how the needs of a community to reuse in safety its heritage can lead to a real regeneration, that allow to revive spaces with an historical-documentary value respecting the atmosphere kept for several centuries, but recalling some important parts that become useful for the current activities.

A deep study was performed on the fortress, that became a valuable opportunity to better understand these kind of buildings, starting from material and techniques used for its realization, and also with respect to the role within the village that hosts it. The purpose has regarded the strengthening of the existing structures, with unconventional interventions, that led to a project involving both architectural and structural needs.

A new image to the fortress was given, by means of low-invasive measures, that allow to connect its different parts, linking the castle to the neighboring territory. A glass cover for the internal courtyard, sustained by a tenso-structure, was conceived. A simplified recursive formula, to define the deformed shape of the steel wire ropes, was developed, simply considering the variation of the angle of each segment and using equilibrium equations to compute the forces acting on them.

The designed intervention have demonstrated that it is possible to preserve the historical heritage without limiting the design possibilities. This way a rediscover of the value of this architectural heritage was allowed, providing a gathering place for the local community exploiting what already exists and keeping alive its cultural value within that province.

REFERENCES


