**Deliverable 9.1**

**Individuation of proper buildings where applying the new technologies**

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WORKPACKAGE 9: Knowledge based assessment

**PROJECT N°:** 244123  
**ACRONYM:** NIKER  
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**COORDINATOR:** Università di Padova (Italy)  
**DURATION:** 36 months  
**INSTRUMENT:** Collaborative Project  
**THEME:** Environment (including Climate Change)
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1 INTRODUCTION

1.1 DESCRIPTION AND OBJECTIVES OF THE WORKPACKAGE

Workpackage 9 is intended to develop and apply knowledge-based assessment procedures to real case studies with the purpose of allowing a final assessment of the proposed monitoring strategies and technologies, strengthening methods and quality assessment approaches. This aim is attained through several tasks involving: (1) The calibration of monitoring systems and devices; (2) The application of the monitoring systems and devices for the identification of the real behaviour of buildings; (3) The application of the knowledge obtained through monitoring to the creation and updating of reliable structural models, to the definition of minimal interventions and to the evaluation of interventions; (4) Based on the previous evaluation, to implement the more optimal intervention techniques to selected case studies; (5) The implementation of instrumented dissipative devices in case studies; (5) Conclusions on the effectiveness of the interventions and the corresponding optimal quality assessment procedures.

In particular, task 9.2, on real case application aims to the application and validation of the monitoring and intervention strategies proposed as a result from previous WP’s as well as task 9.1. For that purpose, a set of case studies have been selected for the application of the methods and techniques. The application of such strategies requires the development of inspection work, structural analyses and monitoring using the tools proposed in previous project tasks. The general process involves as well the proposal of intervention solutions and the selection of an optimal one adequately combining efficiency requirements with satisfactory compliance of conservation principles. The design of the intervention must consider monitoring as an essential tool for the purpose of the controlling and the assessment of effectiveness during and after its implementation. Hence, continuous monitoring is needed before the intervention, for the purpose of inspection and diagnosis, and also during and after the intervention, to allow for long-term verification and quality assessment. Monitoring is also essential to develop sequential interventions where incremental repair or strengthening operations are gradually applied and verified; where the target performance levels are not yet reached, additional incremental operations are applied. Another outcome from previous tasks to be validated through application on case studies is found in the instrumented dissipative devices (task 6.2). This type of device is considered for its application to one or more case studies located in highly seismic risk regions.

1.2 OBJECTIVES AND CONTENTS OF THE DELIVERABLE

The purpose of Deliverable 9.1 on “Individuation of proper buildings where applying the new technologies” is found in the identification and selection of the set of different historical buildings for the application of the proposed monitoring and intervention strategies and technologies. The selection is intended to cover a wide and representative range of conditions regarding geographical location (different countries) local seismicity (low, medium, high), construction material (stone and brick masonry, earth), structural typologies and uses (towers and minarets, churches, large cathedrals, palaces), preservation condition (different levels of damage) and risks involved (i.e., people at risk, valuable artistic contents at risk).

Among the buildings considered are a set of churches in L’Aquila, Italy (Saint Agostino, Saint Marco, Saint Silvestro, Saint Bagio and Giuseppe), a large fortress (Spanish Fortress in l’Aquila), a large Roman construction (Arena in Verona, Italy), masonry towers (Civic Tower in l’Aquila) and minarets (a set of Ottoman minarets in Bosnia), a large Gothic Cathedral (Mallorca Cathedral, Spain), masonry buildings (Mekaad Rasdwan in Egypt, a Medersa in Fez, Morocco, the Int. Conservation Centre in Acre, Israel) and a large earthen construction (Ambel Preceptory in Zaragoza, Spain). All these cases have been carefully analyzed and positively evaluated regarding its potential contribution to the validation of the monitoring systems and model updating strategies resulting from the project. Some of them are also considered for the implementation and validation of intervention techniques resulting from previous WP’s along with the application of the quality assessment procedures.
For the purpose of the selection, all proposed buildings have been evaluated in terms of compliance with the project aims and potential contribution to the main objective of the task, namely the validation of techniques and methodologies. This evaluation has been carried out through an analysis of each building whose result is described in detail in the following sections. The analysis has considered the following criteria:

- **Significance of the building as heritage construction.** Historical, artistic or spiritual significance. Most of the buildings selected are in fact emblematic ones with large significance to local people or even at an international scale. Moreover, almost all of the buildings selected have a religious or civil purpose.
- **Availability of information on the history of the building, including notes on construction technologies and historical events.** In particular, availability of information on historical earthquakes.
- **Local seismicity.** The buildings selected are placed in very different locations concerning the level of seismicity. A few of them are located in low seismic places (such as St. Peter Church in the Czech Republic or the Ambel Preceptory in Zaragoza, Spain) or in medium seismic places (Mallorca Cathedral). However, most of the buildings considered belong to highly seismic regions of the Mediterranean basin (as in the case of the set of buildings in l’Aquila).
- **Present and foreseen future uses and number of people at risk.** While some of the buildings are located in low to moderate seismic regions, all of them show significant risks associated to the present use and the potential number of people (users or visitors) affected by the hypothetical occurrence of an earthquake.
- **Presence of valuable artistic or cultural contents, such as mural paintings or frescoes on walls and vaults.** Some of the selected buildings (as in particular the Mekaad Radwan in El Cairo) actually include valuable decoration and paintings potentially threaded by structural damage caused by earthquakes and other causes.
- **Structural features of the building, including materials, morphology and overall arrangement (structural typologies).** The selection of buildings has been done so as to cover a variety of materials (earth, stone and brick masonry) and structural typologies (as for instance different spans, from small to large ones, roofed with different solutions including timber slabs or masonry vaults).
- **Present condition and damage.** In particular, presence and distribution of deformation and cracking. In some cases, a building located in a low seismic place may be showing a significant or even high risk due to the weakness of materials and structural details, or due to existing damage. The selected buildings show different degrees of damage, ranging from almost intact condition to severely damaged one. Hence, a wide variety of different conditions is considered requiring, in turn, different approaches or methods in the implementation of monitoring and intervention solutions.
- **Previous and on-going studies.** The availability of information previously obtained by means of geometrical survey, inspection, diagnosis, structural analysis or previous monitoring has been considered positively in the selection of case studies. The availability of this information provides interesting opportunities to the calibration of new techniques and approaches by comparison with previous results, while also contributing to an initial more detailed understanding of the building.
- **Previous (already executed) or proposed restoration or strengthening operations.** Similarly, the availability of proposed restoration plans, or the previous implementation of strengthening operations, has been positively considered as it provides an opportunity for the critical analysis and improvement of such actions through the application of the new approaches and methods proposed within the project. Moreover, buildings with planned interventions to be executed during the project period offer important chances for the application and validation of the strengthening and early warning proposed systems. Actually, some of the buildings selected, as some churches in l’Aquila have been chosen
for the real implementation of some of the strengthening and early warning techniques developed in previous WP’s.

In addition to the above items, the evaluation of candidate buildings has also considered a tentative proposal of activities to be developed and the technologies to be applied concerning inspection, monitoring, modelling and structural analysis. The need for seismic strengthening measures has been approximately evaluated and, in some cases, possible specific repair or upgrading solutions have been proposed.

These considerations have allowed, in some cases, a tentative evaluation of the potential contribution of the proposed case study to the project aims. This evaluation has taken into account the following items: (1) Compliance with project objectives; (2) Foreseen opportunities for application of new approaches and technologies (resulting from task 9.1 and previous WP’s; (3) Foreseen opportunities for testing and validating proposed intervention methods; (4) Potential contribution to validate the project’s methods and criteria resulting from WP10.

Developing all the necessary tasks in the different case studies selected will require significant cooperation between different project partners. In particular, narrow cooperation will be needed between those partners involved in the development of new technologies and those providing the case studies for possible application. Although the amount of technological effort invested in each case study will not be the same, for it will depend upon the specific features and opportunities offered, all cases selected will be subjected to a minimum research configuration including the application of at least one or more advanced inspection technologies, some kind of monitoring based on the technologies proposed in deliverable D9.2, modelling and structural analysis, and the proposal of a specific strategy for the upgrading of the building. Monitoring during and after the implementation of strengthening measures and the application of early warning solutions are only considered for a limited number of the proposed case studies.

The buildings finally considered as case studies are listed in tables 1.1 to 1.3. As shown by these tables, the set of buildings chosen cover a wide variety of situations regarding location seismicity, structural types, people and artistic contents at risk and damage condition. As shown by table 1.1, buildings located at very different seismicity levels are considered, with a majority of buildings placed at moderate to highly seismic places, while also including a limited number of cases in low seismic regions. In particular, all the different structural typologies considered are well represented by cases situated in moderate to high seismic places. Regarding the risks involved, the set of buildings selected also cover a variety of situations involving moderate, intermediate and high risks related to people and also and artistic or cultural contents. In particular, there are buildings located at highly seismic places involving very important risks for people and artwork. Finally, table 1.3 categorizes the cases selected according to the damage level observed at present. Again, the cases selected show a wide variety of conditions. Not surprisingly, the level of existing damage increases with the seismicity of the location, although there are also examples of moderately to severely damaged buildings in low seismic places.
Table 1.1 - Categorization of case studies according to structural types and seismicity.

<table>
<thead>
<tr>
<th>SEISMICITY / TYPOLOGY</th>
<th>Low to moderate seismicity (a/g &lt; 0.10)</th>
<th>Intermediate (0.10 \leq a/g &lt; 0.20)</th>
<th>High seismicity (a/g \geq 0.20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Houses, palaces</td>
<td>Ambel Preceptory (Zaragoza) (earthen)</td>
<td>Mekaad Radwan (El Cairo) Medersa (Fez)</td>
<td>Int. Conservation Center (Acre) Building of St. Sacramento Monastery (Lisbon)</td>
</tr>
<tr>
<td>Churches and cathedrals</td>
<td>St. Peter Church (Prague) Mallorca Cathedral</td>
<td>St. Agostino (L'Aquila) St. Marco (L'Aquila) S. Silvestro (L'Aquila) St. Biagio &amp; S. Giuseppe (L'Aquila)</td>
<td></td>
</tr>
<tr>
<td>Fortresses</td>
<td></td>
<td>Spanish Fortress (L'Aquila)</td>
<td></td>
</tr>
<tr>
<td>Towers</td>
<td>Bell tower of St. Vitore (Arcisate)</td>
<td>Civic Tower (L'Aquila)</td>
<td></td>
</tr>
<tr>
<td>Minarets</td>
<td>Ottoman Minarets (Bosnia)</td>
<td>Ottoman Minarets (Bosnia)</td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td>Arena (Verona) Arche Scagliere (Verona)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 1.2 - Categorization of case studies according people at risk and seismicity.

<table>
<thead>
<tr>
<th>SEISMICITY / PEOPLE AT RISK</th>
<th>Low to moderate seismicity $a/g &lt; 0.10$</th>
<th>Intermediate $0.10 \leq a/g &lt; 0.20$</th>
<th>High seismicity $a/g \geq 0.20$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moderate limited number of users or visitors</td>
<td>Ambel Preceptory (Zaragoza)</td>
<td>Mekaad Radwan (El Cairo) Medersa (Fez) Int. Conservation Center (Acre) Building of St. Sacramento Monastery (Lisbon) Ottoman Minarets (Bosnia) Arche Scagliere (Verona)</td>
<td>Spanish Fortress (L’Aquila) Ottoman Minarets (Bosnia)</td>
</tr>
<tr>
<td>High congregations High risk to nearby buildings / pedestrians</td>
<td>St. Peter Church (Prague)</td>
<td>Bell tower of St. Vitore (Arcisate)</td>
<td>St. Agostino(L’Aquila) St. Marco(L’Aquila) S. Silvestro(L’Aquila) St. Biagio and S. Giuseppe (L’Aquila) Civic Tower (L’Aquila)</td>
</tr>
<tr>
<td>Very high Large number of visitors, spectacles</td>
<td>Mallorca Cathedral</td>
<td>Arena (Verona)</td>
<td></td>
</tr>
</tbody>
</table>
Table 1.3 - Categorization of case studies according to damage condition and seismicity.

<table>
<thead>
<tr>
<th>SEISMICITY / DAMAGE</th>
<th>Low to moderate seismicity $a/g &lt; 0.10$</th>
<th>Intermediate $0.10 \leq a/g &lt; 0.20$</th>
<th>High seismicity $a/g \geq 0.20$</th>
</tr>
</thead>
<tbody>
<tr>
<td>No significant damage /</td>
<td>St. Peter Church (Prague)</td>
<td>Medersa (Fez)</td>
<td>Bell tower of St. Vitore (Arcisate)</td>
</tr>
<tr>
<td>Moderate damage</td>
<td>Mallorca Cathedral</td>
<td>Int. Conservation Center (Acre)</td>
<td>Arche Scagliere (Verona)</td>
</tr>
<tr>
<td>Severe damage</td>
<td>Ambel Preceptory (Zaragoza)</td>
<td>Mekaad Radwan (El Cairo)</td>
<td>S. Silvestro (L’Aquila)</td>
</tr>
<tr>
<td>Partial collapses + emergency stabilization</td>
<td>Arena (Verona) (historically)</td>
<td>St. Agostino(L’Aquila)</td>
<td>St. Marco(L’Aquila)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>St. Biagio and S. Giuseppe (L’Aquila)</td>
<td>Spanish Fortress (L’Aquila)</td>
</tr>
</tbody>
</table>
2 PROPOSED CASE STUDIES

2.1 TRADITIONAL STONE MASONRY MINARETS IN BOSNIA AND HERZEGOVINA

2.1.1 General description

The subject of the case study is the seismic behavior of traditional stone masonry minarets dating from the Ottoman period in Bosnia and Herzegovina, 15th to 19th century. This type of minaret is almost regularly attached to the mosque building. One complete study should involve both minaret and the mosque building as the unique structure. However, for the sake of simplicity and a stepwise approach this study is restricted to the minarets only with a simulated attachment to the building.

The goal of the study is to provide a stepwise procedure for future seismic assessments. The procedure comprises firstly the simplified analysis, then estimating the level of confidence in the results obtained and finally performing a complex 3D analysis if found to be required. The suggested procedure based on the experience gained during the post-war reconstruction in Bosnia and Herzegovina (after 1995) will be verified by this study involving simplified and complex seismic analysis as well as the on-site dynamic characterization of a representative minaret structure.

The relevant input information has been prepared for nineteen minarets in order to estimate the relevancy of the differences with respect to seismic assessment. The intention is to gather information on more minarets respecting the time-plan of activities. Three of the nineteen minarets are tentatively available for on-site dynamic testing. Official consent is required for that (those) finally chosen for testing. One of these three is strengthened with five vertical CFRP strips. Additionally, a long-term monitoring by using innovative devices might be considered as well.

For further detailed description within this report, the minaret of the Hadži-Alija’s mosque in Počitelj (Figure 2.2) has been chosen as the representative structure with notes referring to the other minarets and indicating relevant differences.
2.1.2 The Hadži-Alija’s mosque. Name, location and description

The Hadži-Alija's or Šišman Ibrahim-paša mosque is a constituent part of the historical Old town Počitelj (Municipality Čapljina), situated approximately 30km south from the city of Mostar. It stands on cadastral plot 2655, cadastral municipality Počitelj, and is owned by the Islamic Religious Community.

The mosque was built in 1562-63 in a classical Ottoman style of single-room domed mosques in Bosnia and Herzegovina. The enclosed area is 12x12m roofed by a hemispherical dome with its top at 15m above the floor level. The entrance porch is approximately 4m wide and covered with three smaller domes. The stone minaret of 32,83m height stands to the right of the entrance (Figure 2.3 to Figure 2.5). There was formerly a wooden canopy supported on posts in front of the sofa, which was removed in 1952.
The transition from the walls to the dome was achieved structurally by means of tromps and eight pendentives ending in a drum. The exterior sofas have four round stone columns between them and are joined to the front wall with pointed arches, supporting three domes on low octagonal bases.

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1 Institute for protection of cultural heritage Mostar (2002). Design for the reconstruction of the Hadžić-Alija’s mosque in Počitelj (rearranged drawing).

2 Ibid.

3 Ibid.
drums over pendentives. The walls of the building are made of symmetrical ashlars, and are some 110 cm thick. The exterior façades are stone, and the walls are plastered on the interior. The dome and minaret cone are clad with sheet lead.

The central interior space beneath the dome has a stone mihrab, a mimber (pulpit) and a mahfil (gallery) to the right of the entrance, of classical style. The stone portal has door-jambs with an arched opening for the door, a triangular finishing above the door within a moulded rectangular frame.

The decorative elements of the building are of classical style: stalactite-like ornaments in the squinches (Figure 2.6), on the capitals of the columns, beneath the balcony of the minaret and in the mihrab; decorative pierced panels in the window arches; moulded and bas-relief ornamentation on the minber, mihrab and mahfil. The wooden door of the portal is decorated with a geometrical design.4

2.1.3 Historical note

It is supposed that the fortified town and its attendant settlements were built by Bosnia's King Stjepan Tvrtko I in 1383. The walled town of Počitelj evolved over the period from the sixteenth to the eighteenth century. Architecturally, the surviving stone-built parts of the town are a fortified complex, in which two stages of evolution may be observed: medieval, and Ottoman.

The first documented reference to the town dates from 1444, in Charters issued by Kings Alfonso V and Friedrich III. During the period 1463-1471 the town held a Hungarian garrison. Following a brief siege in 1471, the town fell to the Ottomans, and was to remain within the Ottoman Empire until 1878. From 1782 to 1879 it was the centre of a kadiluk (area under the jurisdiction of a kadija or qadi - judge) and from 1713 to 1835 it was the headquarters of the Počitelj military district.

With the establishment of Austro-Hungarian rule in BiH in 1878, Počitelj lost its strategic importance and began to deteriorate rapidly. It was in this period that a section of the ramparts was pulled down at Donja kapija or Lower Gate and the road to Donje polje was laid. Later a further section of the ramparts was demolished, together with the smaller gate behind the hamam, and the road to Gornje polje was widened, thus destroying the continuity of the Počitelj ramparts. The loss of the town's strategic role helped to safeguard the original urban architectural ensemble, so that the town has been preserved in its original form to this day.

According to its chronogram, the mosque was built in 970 AH (1562-63 AD) by Hadži Alija. Over the period 1471-1698 besides the mosque, other buildings typical for the Ottoman period were erected in the area: mekteb (Muslim primary school), imaret (charitable kitchen), medresa (Muslim high school), hamam (Turkish bath), han (inn) and sahat-kula (clock-tower).

Since 1955, within the jurisdiction of the Institute for the Protection of the Cultural and Historical Heritage of SRBiH, some restoration work has been carried out in a number of stages: replacement of the lead sheeting on the domes, restoration of stone window transomes, etc.

The Šišman Ibrahim-paša or Hadži Alija's mosque was protected as a cultural monument pursuant to a Ruling by the Institute for the Protection of Cultural Monuments of SR BiH, No.: 02-673-3/62 dated 23 October 1962. The interior was painted, with a new composition by Nihad Bahtijarević completed in 1988.

The mosque was blown up in 1993: the dome and minaret were demolished, and the rest of the building was badly damaged (Figure 2.7). The Plan for the permanent protection of the historic urban site of Počitelj provides for the rehabilitation of the mosque of Hadži Alija as a priority. On 5 August 2001 an experts group appointed by the Government of the Federation, pursuant to its resolutions of 24 November 2000, set out the programme guidelines for the rehabilitation of the

4 Commission to preserve national monuments in Bosnia and Herzegovina (2002). Decision on designation of The historic urban site of Počitelja as a National Monument of Bosnia and Herzegovina.
Počitelj mosque, which is being carried out according to the renovation project drawn up by the Institute for the Protection of the Cultural, Historical and Natural Heritage, Mostar, no. P-02/2002, in May 2002.

The Federal Ministry of Urban Planning and the Environment issued a Ruling (No. UPI/02-23-2-47/02, 17 June 2002) on the rehabilitation of Hadži-Alija/Šišman Ibrahim-paša's mosque in the historic site of the town of Počitelj, (cadastal plot no. 2665, cadastral municipality Počitelj, Municipality Čapljina, with a total area of 659sq.m., owned by the Islamic Religious Community's Vakuf Commission, Počitelj.). The reconstruction was performed during 2002 and 2003 and the building officially reopend on October, 10th 2003. Reconstruction comprised total reconstruction of the minaret, central dome and the small domes above the porch, closer to the minaret and two out of four columns supporting the porch structure. A half of the middle small dome above the porch was reconstructed as well. The walls, drum and other structural elements were repaired.

2.1.4 Historical and artistic significance

The Hadži-Alija's mosque is a monumental building by itself but at the same is a part of the historic ensemble comprising fortifications, public and residential buildings. Over the medieval and Ottoman period the site experienced a noticeable growth due to its significant strategic position. The historical circumstances changed after 1878 (the end of Ottoman ruling) in sense that Počitelj lost its strategic importance. The loss of the town’s strategic role helped to safeguard the original urban architectural ensemble, so that the town has been preserved in its original form to this day.

2.1.5 Structural and material features

2.1.5.1 Geometrical characteristics

The minaret is a tube-like slender structure, supported vertically at the foundation level and horizontally at the bottom part by the mosque building. The height of the structure is 32,83m excluding the extension of 1,3m for the finishing copper ornament - the finial. The horizontal cross-section is a ring with a 14-sided polygon outside and a circle inside. For further information on geometry, the outer polygon is replaced with its inscribed circle. The inner spiral staircase is starting from 40cm above the ground level and ending at the sherefe (balcony) at approximately 22,4m above the ground level. The inner tube containing the spiral staircase is of the same

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5 Ibid.
diameter over the whole staircase height, while the outer side is narrowing step-by-step towards the top. Outer diameter of the base has a constant value of 2,62m, of the shaft - 1,92m and of the upper shaft - 1,755m. Since the inner tube is constantly of the same diameter (1,48m), the thickness of the perimeter wall changes respective to the outer diameter (570, 220 and 138mm) (Figure 2.8 to Figure 2.11).

Six typical portions (A to F) may be noted across the height, starting from the ground level: The lowest part is the base of 7,90m height. The outer diameter is 2,62m, the inner 1,48m and the thickness 570mm. The inside contains the stone spiral staircase starting approximately 400mm above the ground level. The base ring is weakened by two door openings, one at the ground floor level and the other at the level of the mosque’s gallery. This is the most common construction of masonry minarets in Bosnia and Herzegovina but the following alterations may be noticed:

In rare cases the whole minaret is of a squared horizontal cross-section, both outside and inside. The structure of these minarets is similar to the structure of clock and bell towers and it is not the subject of this case study. In some cases due to terrain morphology or functional reasons, the inner staircase is starting from a higher level instead of just above the ground. In these cases the part below the staircase has an infill typically constructed with rubble masonry with weak mortar. The outer leaf is constructed with ashlars. The horizontal cross-section of this solid part is commonly squared.

In some cases the base is of a squared outer shape with the inner staircase starting either just above the ground level or at a higher level due to functional reasons. If the staircase starts at higher level then there is just one door opening within the base ring, leading to the mosque’s gallery.

The transition from the base outer diameter of 2,62m to the shaft outer diameter of 1,92m is constructed over the height of 1,93m. With the inner diameter of 1,48m the wall thickness ranges from 570mm to 220mm. This part is called the shoe. The inclination of the transition is 5,24:1 or 79,2 degrees with respect to the vertical cross-section. The possible alteration are the following:

In modest constructions, there is no inclined transition and the shaft is simply emerging from a wider base which is here usually of a squared shape. The corners of the top of the base are cut for artistic reasons.

The central part is the shaft with the outer diameter of 1,92m, the inner 1,48m and the thickness of 220mm. The height of shaft is 11,44m and it goes to the bottom level of balcony structure at 21,26m above the ground. The outer side, for the sake of simplicity replaced with the circle, is actually a 14-sided polygon.

Within the studied group the polygon is 12, 14 or 16-sided and the thickness of the shaft varies between 200 and 400mm.

The next part up is the balcony (sherefe), starting from the top of the shaft and ending at 22,37m above the ground, which is the floor level of the balcony and also the ending point of the inner spiral staircase. Thus the height of the balcony structure is 1,09m, except for the perimeter fence. The outer diameter at the balcony floor level is 2,87m. Additionally, there is a perimeter fence constructed with stone slabs of approximately 50mm thickness. The outer surface of the sherefe is shaped in a form of stalactite ornament which is considered as piece of art and requires a very skilled work.

The first part above the balcony is called the upper portion of the shaft. Its bottom part is weakened by the door opening serving as the exit to the balcony level. The outer diameter is 1,755m, the inner diameter 1,48m and the thickness 138mm. The height of this part is 5,21m. The space inside contains only the central timber mast of 200mm diameter, that supports the upper finishing structure of the hood.

The finishing part is the conical hood of 5,26m height. The structure of the hood is a timber structure constructed around the central mast. Wooden planks are nailed to the substructure and covered with lead sheets. In other cases final cladding may be constructed with shaped stone slabs additionally fixed with outer iron hoops. A copper decoration, called finial is placed on the top
of the hood. The height of the finial is approximately 1,30m and its top is 34,13m above the ground level.

The inner stone spiral staircase is a distinctive element of the structure. Ninety steps are covering the height of 21,99m, giving the average step height of 244mm. The steps are shaped and constructed in such a way that they form the central column of 220mm diameter. With the overall inner diameter of 1480mm, the remaining passing width is 630mm. The width achieved this way provides a support for the next upper step. Thus the steps supported by the step below and the central self-forming column and approximately 100mm into the perimeter ring.

The minaret is founded on the rock approximately 400-500mm below the ground level. The foundation is of a square shape and consists of masonry block joined by mortar and iron cramps.

2.1.5.2 Construction details

The perimeter wall is constructed as a three-leaf wall at the base (Fig.12) and as a single-leaf wall at upper parts, except for the final hood which is done as a timber structure.

The outer leaves of the base are constructed with ashlar stone and thin-layer lime mortar joints. The inner core is made of rubble stone and lime mortar. The outer leaf at the inside is done with ashlars shaped to form the tube for the inner spiral staircase. The joints at the inside vary from a thin-layer joint to a standard one.

The single-leaf perimeter wall of the shaft, 220mm thick and the upper shaft, 138mm thick is done with ashlars with thin-layer lime mortar joints. The outer side is finely shaped forming a polygon and the inner side is less finely shaped forming a circle. Ashlars are set in clearly distinctive rows - rings of approximately 300mm height. Fourteen ashlars in each ring are connected on the upper side with iron cramps set in the prepared holes filled with lead. Vertical joints are staggered in the following rings. Adjacent horizontal rings are not vertically connected by any additional elements except with the thin-layer mortar joint.

The shoe (transition) and the sherefe (balcony) are specific parts because of their inclined outer side and because they decorate appearance. The sherefe has a sophisticated stalactite decoration. The outer surface of the shoe is formed by triangles. The part of the base just below the shoe and the part of the shaft just above the shoe are decorated as well.

The spiral staircase is formed by staggering steps (Figure 2.12). Each step consists of two stone blocks with a space between them filled with rubble stone in lime mortar. One full step is forming a central angle of approximately 77 degrees. Only one third of the full step is inserted approximately 10cm into the perimeter wall. The next step is overlapping approximately two thirds of the step below. Iron cramps are placed on the top and fixed in the prepared holes filled with lead. The described construction is very often, but in other minarets one can also find that the full step is narrower and done with a single stone block. Steps are vertically connected with an iron dowel at the position of the self-formed central column.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

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Figure 2.8 - Horizontal cross-section at the level of the upper part of the shaft (Part E).

Figure 2.9 - Horizontal cross-section at the level of the shaft (Part C)

Figure 2.10 - Vertical section.

Figure 2.11 - Horizontal cross-section at the level of the base (Part A).

6 Ibid.
2.1.5.2.1 Materials

The stone used for minaret perimeter walls and steps is a sort of limestone locally called *Tenelija*, which can still be obtained from the quarry called Mukoša, in Mostar area. Tenelija was also used for the construction and the reconstruction of the Old Bridge in Mostar. Within the same quarry one can also find a similar type called *Miljevina* which is softer and more porous than Tenelija. Cutting of Tenelija is rather easy and therefore it is mostly used for ashlars for various representative structures (bridges, religious buildings etc.) (Figure 2.13).

Tenelija is oolithic limestone, uniformly granulated (visible grains), ivory-whitish coloured with characteristics: porosity 23%, specific density 27kN/m³, wet density 21kN/m³, dry density 19kN/m³, moisture content 8%, uniaxial compressive strength 20N/mm² and E modulus 13000N/mm².⁷

Types of stone used for other minarets in Bosnia and Herzegovina, come from closest appropriate local quarries.

A for the binder, hydrated lime mortar is typically used for construction of minarets as well as other historical buildings in the area.

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2.1.6 Present and foreseen future use. People at risk.

After the reconstruction and reopening in October 2003, the mosque has its sacral function and it’s opened for tourist visit as well. The same usage is expected in the future, with tendency of an increased number of visitors. Larger number of local people is gathering Fridays for the noon prayer, then twice a year on the occasion of Eids and also on other sacral occasions. During the tourist season (June-September) the mosque as well as the whole Old town Počitelj is visited by many tourist groups, who make a stop on their way from and to the Adriatic sea. As a reference the approximate number of visitors to the nearby City of Mostar is 20,000 annually, staying in average two days in the area.

The minaret is the most vulnerable part of the mosque complex so that the area around the minaret, including the mosque building is the area of risk.

2.1.7 Considerations on valuable cultural contents

The minaret itself has no fixed artistic contents. The value of mosque building’s interior painting will be explored.

2.1.8 Present condition and damage

2.1.8.1 Considerations on the present condition

While the minaret was completely reconstructed in 2003, the present condition may be considered good and without damages.

2.1.8.2 Main observed damage and decay

There is the information that the minaret was in a rather bad condition before 1990. The process of exploring this information is on-going.

2.1.9 Information on local seismicity

2.1.9.1 Local seismicity

Bosnia and Herzegovina is situated in active seismic region of South-East Europe. This was confirmed by few strong earthquakes in the second part of the last century. With respect to the 500 years return period, most of the country is within seismic zones of 7th and 8th intensity degrees according to MCS-scale or the new European Macroseismic Scale EMS. Smaller parts correspond to the 9th seismic intensity zone. Thus in most case the peak ground acceleration (PGA) between 0.10 and 0.20 g needs to be considered and in a few even PGA of 0.30-0.35 g.

The list of recorded earthquakes in the region in the period 1900-2009 is provided from the Institute for hydrology and meteorology, see Figure 2.14. The list contains 1606 records with date and time, location of epicenter, depth of the hypocenter and magnitude. The distribution of the recorded earthquakes with regard to their magnitude is: 0.56% of magnitude 6 to 6.6, 2.79% of magnitude 5 to 6, 23.13% of magnitude 4 to 5, 71.47% of magnitude 3 to 4 and 1.05% of magnitude 2 to 3. The data will be further sorted, grouped, described and presented on the appropriate map.

The area of Mostar is generally in the 8th seismic zone, corresponding to the peak ground acceleration of 0.2g.

8 Šaravanja K. & Čolak I. Stone for the reconstruction of the Old bridge in Mostar. (SEM images from LGA report on the materials for the reconstruction of the Old bridge in Mostar)
2.1.9.2 Characterization of the seismic action

Dynamic characterization, by ambient vibration testing was performed for the Karadjozbey’s mosque and its minaret in 1991 by the Institute for seismic engineering from Banja Luka, Bosnia and Herzegovina.9 The Karadjozbey’s mosque is the main mosque in Mostar, with a very similar minaret structure to the minaret of the Hadži-Alija’s mosque in Počitelj.

Piezoelectric accelerometers “Bruel and Kjaer 8306” were used for measuring ambient vibrations. The characteristics of the instruments used are: sensitivity 1000pC/m/s², frequency domain 0,2-1000Hz, mass 500g and sensitivity 2e-6. The obtained data were analyzed by the two-channel spectral analyzer Hewlett-Packard HP 3582 A.

Nine accelerometers were position over the minaret height up to the level of the sherefe (balcony). Measuring was performed in two directions: perpendicular and parallel to the mosque building.

The eigen frequencies calculated for the perpendicular direction were: for the first mode 1,2Hz, for the second mode 5,6Hz and for the third mode 11,4Hz. Damping ratios are 3%, 3% and 4% respective to eigen modes. The second eigen frequency of the minaret is approximately equal to the first eigen frequency of the mosque building (6,0Hz). The difference between the minaret part attached to the building and the free part are noticeable on plot of eigen vectors.

For the parallel direction the following eigen frequencies were obtained: the first 1,20Hz, the second 5,4Hz and the third 11,4Hz. Damping ratios are 5%, 4% and 4% respectively. The second frequency is again similar to the first eigen frequency of the mosque building (6,0Hz).

Figure 2.14 - Location of the epicenter of earthquakes recorded in the area of Bosnia and Herzegovina in the period 1900-2009 (Interprojekt 2010).

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2.1.10 Previous and on-going studies

2.1.10.1 Inspection works

2.1.10.1.1 Geometric survey
The reconstruction was performed according to plans based on the geometry of the original structure. There is no record on checking the geometry after the reconstruction.

2.1.10.1.2 Research on materials
Since the same stone (Tenelija) was used for the reconstruction of the Old bridge in Mostar, a thorough investigation was performed before and also after the reconstruction of the bridge.

2.1.10.2 Structural analysis
A spreadsheet structural analysis for the reconstruction was done in 2002, taking into account vertical (self-weight) and horizontal (wind, seismic) action. The system was considered elastic. Horizontal joints were analyzed assuming no-tension capacity, redistribution of stresses across the remaining compressed area and limited compressive (crushing) strength.

2.1.11 Previous or on-going restoration works

2.1.11.1 General restoration works
The minaret was completely reconstructed in 2003. There is no information of any restoration works afterwards.

2.1.11.2 Actions oriented to seismic retrofitting
Five vertical CFRP strips were planned at the inside of the perimeter wall in order to cover the calculated tension and prevent from excessive horizontal joint opening. The solution was implemented and five vertical stripes placed by a licensed company. Carbon fiber strips used are four Sika Carbodur S812 and one Sika Carbodur S512 (five S812 planned), each of them approximately 13.20 m length. The strips cover the upper fourth of the base, the whole transition and the lower 81% of the shaft height (Figure 2.15 to Figure 2.17).

![Figure 2.15 - Horizontal cross-section with the designed arrangement of vertical CFRP strips (Interprojekt, 2003).](image1)

![Figure 2.16 - Void in the step left for later mounting of CFRP strip (Interprojekt, 2003).](image2)
2.1.12 Historical research

The information on the damaged status of the minaret of the Hadži-Alija’s mosque before 1990, will be explored within the study. Information on the geometry and materials of other traditional mosques in Bosnia and Herzegovina, will be searched respecting the time-plan of the project activities.

2.1.13 Local seismicity and characterization of seismic action

Further sorting and interpretation of records on regional seismic activities will be done and processed along relevant information from codes and guidelines. Dynamic characterization needs to be obtained by ambient vibration testing. Depending on the possibilities, it would be better to perform testing at the retrofitted minaret of the Hadži-Alija’s mosque in Pocitelj and at similar but un-retrofitted minaret in Mostar.

2.1.14 Proposed (additional) inspection works

2.1.14.1 Inspection works.

2.1.14.1.1 Geometric survey. Aims, methods and technologies

Brief checking of the existing geometry will be performed and data compared with reconstruction drawings. The checking will be performed by means of simple laser measuring devices and manually.

2.1.14.2 Research on materials. Aims, methods and technologies

No further stone testing is predicted. The existing information will be sorted and commented. Depending on the possibilities and for the sake of full information, chemical investigation of the mortar should be performed.

2.1.14.3 Other inspection works

Dynamic characterization needs to be obtained by ambient vibration testing. Depending on the possibilities, it would be better to perform testing at the retrofitted minaret of the Hadži-Alija’s mosque in Pocitelj and at similar but un-retrofitted minaret in Mostar.

2.1.15 Proposed structural analysis. Aims and methods

2.1.15.1 Modeling and structural analysis methods
Existing spreadsheet structural analysis needs to be updated and possibly new simplified methods developed. Modeling of the minaret along with the inner staircase should be done, and appropriate 3D dynamic analysis (FEM or other) performed. The analysis should be performed for two general cases: un-retrofitted and retrofitted minaret structure. Wind load needs to be considered as alternative horizontal action. Simplified and complex analysis need to be compared in order to estimate the limitations and the level of confidence in simplified analysis as less costly and quicker assessment tools.

2.1.15.2 Model updating and monitoring. Interaction with monitoring
The developed 3D model should be updated based on the results of ambient vibration testing. Simplified model(s) should be updated based on the results of ambient vibration testing and updated 3D analysis.

2.1.15.3 Assessment of possible intervention methods
Possible intervention methods will be discusses based on the result of structural analyses and ambient vibration testing.

2.1.16 Envisaged interventions

2.1.16.1 Considerations on the need for conservation, repair and maintenance
Possibility of installing innovative monitoring and early-warning devices should be considered in order to provide better understanding of the behavior of the structure and prompt alert for repair or maintenance.

2.1.16.2 Considerations on the need for seismic strengthening
One strengthening method has been implemented and needs to be discussed along with other alternative methods, based on the results of the study.

2.1.17 Potential contribution to the project

2.1.17.1 Compliance with project objectives
The aim of the study is to provide modern and sound design methods for knowledge based assessment of the specific tower-like monumental structure with the final objective to enable cost-efficient and reliable mitigation of potential damages to CH asset caused by earthquakes.

2.1.17.2 Potential contribution to validate the project’s methods and criteria
Possibility for applying early warning innovative devices may be considered.

2.1.17.3 Possibility of monitoring long-term effectiveness of intervention
Possibility for applying innovative devices for monitoring long-term effectiveness of the existing retrofitting intervention (vertical CFRP strips) may be considered.

Table 2.1 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td>Information on geometry exists. On-site checking by using simple laser measuring devices and</td>
</tr>
</tbody>
</table>
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<table>
<thead>
<tr>
<th>1.2 Materials</th>
<th>Information on stone testing exists.</th>
<th>Chemical analysis of mortar.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.3 Internal morphology</td>
<td>No testing performed.</td>
<td>No testing required for the study.</td>
</tr>
<tr>
<td>1.4 Other</td>
<td>Dynamic characterisation by ambient vibration testing performed in 1991 for a similar minaret structure.</td>
<td>Ambient vibration testing, possibly both for unretrofitted and retrofitted structure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>2. Monitoring</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 Static monitoring</td>
<td>-</td>
<td>Nor planned, but may be considered as an option.</td>
</tr>
<tr>
<td>2.2. Dynamic monitoring</td>
<td>-</td>
<td>Not planned, but may be considered as an option.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>3. Structural analysis</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1 Modeling and model updating</td>
<td>Simplified spreadsheet modeling exists.</td>
<td>Updating of the existing simplified modeling and possibly development of other simplified models. Developing the sophisticated 3D model including the inner spiral staircase and retrofitting. Updating both models based on the results of ambient vibration testing.</td>
</tr>
<tr>
<td>3.2 Seismic analysis</td>
<td>Simplified seismic analysis performed</td>
<td>Updating of the existing simplified analysis and possibly development of other simplified analyses. Performing the 3D FEM (or appropriate) analysis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>4. Intervention</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</td>
<td>Design of strengthening with CFRP strips performed for two minarets.</td>
<td>Critical review of the implemented retrofitting method and possibility for implementing other methods based on the results of the study.</td>
</tr>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.2 Real implementation</td>
<td>Retrofitting with CFRP strips implemented at the inside of the perimeter wall.</td>
<td>Commenting the implemented retrofitting based on the results of the study.</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td>-</td>
<td>Not planned, but may be considered as an option.</td>
</tr>
</tbody>
</table>
2.1.18 References


2.2 RAS CHERRATINE MEDERSA, FEZ MEDINA, MOROCCO

2.2.1 Name, location and description

Fez medina, the spiritual and cultural city and a listed UNESCO World Heritage site, was founded in 789. Its geographic location on the fertile Saiss Plain, in the hollow of a valley, made it an economic, political, and strategic crossroads, and was a decisive factor in the city’s development and influence. The medina is characterized by the subtle sumptuousness of its palaces, the richness of its museums, the solemn grandeur of its medersas, and the magnificent profusion of its mosques.

The whole of the old city, (Fez-El-Bali in Arabic), is nothing less than a great open-air museum, with countless masterpieces of architecture awaiting your discovery in its 9,400 narrow little streets. Among the 10,572 listed buildings of historical interest it contains, Fez boasts 185 mosques, numerous medersas, and a selection of truly magnificent palaces. Note that the foundation of the Al Quaraouiyine University mosque was in 857 and marked the start of the city’s Golden Age.

The medersas (Koranic universities or Islamic educational institutions) bear living witness to the city’s intellectual and scientific past. They are the work of the Merinid sultans, and played a prominent part in political, educational, and cultural life. Founded in the 13th century, the first to be built was the Seffarine Medersa, outstanding for the restrained elegance of its decor. The Bou Inania Medersa is remarkable for the sumptuousness of its architecture - its wealth of sculpted stuccowork and carved cedar wood, and its rich onyx and marble decor. Ibn Khaldoun taught there. The El-Attarine Medersa was built in 1325 and is one of the wonders of the city, a true masterpiece, its decor - sculpted marble and calligraphy, cedar wood arches, finely worked mosaics and arabesques - of breath-taking delicacy. The Ras Cherratine Medersa was built by Sultan Moulay Rachid in the 17th century, and could accommodate up to two hundred students at a time, see Figure 2.18.
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Figure 2.18 - Different views of the medersa.
Figure 2.19 - Main façade (top) and architecture plans: ground floor, first floor (middle) & second and terrace ceiling (bottom).
2.2.2 Historical note

The Ras Cherratine Medersa, a national monument, is the largest medersa in Fez built by Sultan Moulay Rachid in 1670. It was a residence and study space for students who attended the nearby Al Quaraouiyine University.

2.2.3 Historical and artistic significance

The medersa represents not only one of the architectural characteristics of Fez, but also reflects the interest granted to the students and the Soufi movement in the imperial city.

2.2.4 Structural and material features

The Ras Cherratine Medersa structure is a masonry construction of fired bricks with 3 levels, the thicknesses of the front walls range from 60-80 cm, while that of the inner wall is 40 cm. The ground floor height is of 4.8 m, while the 1st and 2nd floor is 3.0 m. The overall building height is 12 m. The geometry is described in the architectural plans, see Figure 2.19. The floor diaphragms are made of wooden beams and the type of wood commonly used is cedar.

2.2.5 Present and foreseen future use. People at risk.

The Ras Cherratine Madrasa has been recently restored by the Holding company Al Omran, under the agreement between the Ministry of Culture and the Ministry of Housing, Urbanism and Spatial Planning. It gives new life to this prestigious institution, which becomes a nice place of education in the fields of heritage and architecture. Indeed, part of the Madrasa will be used by the Ecole Nationale d'Architecture, Rabat, as a space for teaching and research, open to architects and other highly qualified specialists.

2.2.6 Considerations on valuable cultural contents

It has some beautiful carved plaster and woodwork (Figure 2.20) and three fantastic sets of double balconies, spanning all the three floors in each corner, a fourth corner has a wash room. It must have been a magnificent place in its prime. It has been recently restored and this is a good thing, however the restored work does not match the attention to detail in the finishes as the original work.
2.2.7 Information on local seismicity

The geological position of Morocco at the encounter of several interacting tectonic plate is the raison that, historically, Fez city was repeatedly destroyed by several strong earthquakes. From the 9th to the 11th century, the earthquakes were described in a brief way: the writings referred to the destruction of constructions without other precise details. The description became relatively more detailed but the objective remained purely informative. It was at the 17th century that one found the details of the 1624 and 1663 earthquakes reported in a particular mail: the damages which affected the buildings in Fez were mentioned in a detailed way. The extent of the destruction of the 1755 seism, Lisbon earthquake, was mentioned by several sources: the destruction concerned even the palaces and the mosques in Meknes while the damage was less impressive in Fez.

In addition, a chronology of the great earthquakes in Morocco since year 881, is presented according to La Grande Encyclopédie du Maroc (1987):

- In September 22, 1522, Fez and the villages of the surrounding areas were completely destroyed.
In January 26, 1531, a violent seism was felt in Morocco but this event is unknown in Moroccan literature.

In May 11, 1624, a catastrophic seism destroyed most of the Moroccan towns including Fez and Meknes. This earthquake is relatively well documented.

In November 1, 1755, the seism which struck Lisbon destroyed the majority of the Moroccan coastal towns from Tangier to Agadir. At this epoch the large cities were especially located inside of the country like Fez, Meknes and Marrakech. One of the well documented destructive tsunamis in the Atlantic Ocean. The Arabic and European archives deal us with so important information concerning the event.

About 26 days after Lisbon earthquake, a seism more stronger and violent affected city of Fez after the last prayer of the day: many minarets and mosques were destroyed in different places in the city and about 10 000 persons were killed.

In April 12, 1773, a devastating seism almost destroyed all the town of Tangier and several houses in Fez have collapsed.

In January 4, 1929, an earthquake caused damages in Fez and villages of the surrounding area.

What is interesting in this chronology of the major earthquakes in Morocco, it is the frequency of the earthquakes which struck the town of Fez (1046, 1079, 1408, 1522, 1623, 1624, 1708, 1755, 1773, 1776, 1867), so much that its inhabitants believed that the ground of their city often moved because it was hollow and full of subterranean water.

The medina like Fez is regarded as a good example of the experiment of the old frame face to the earthquakes. In addition to the compact architecture of the old frame, the knowledge of the local construction practices is fundamental and should guide the choice for successive restoration and rehabilitation operations. The principal building materials used in the construction include the hearth blocks, as well as plaster, wood, and stone. Traditional ceramic tiles, zelij, or cedar woodwork are the common finishing materials. Indeed, one of the constructive typologies of the structure load-bearing wall is the masonry carried out with earth bricks, and bound by a loam mortar or lime, between which are intercalated wood elements of cedar. This provision of two materials, one rigid and the other flexible device, allows absorption of the seismic horizontal loads.

The renovation and restoration work on the other hand has been done with mostly traditional materials and methods of construction. Local masons and craftsmen do the majority of the work.
Figure 2.21 - Seismic zones (5 zones instead of 3 zones in the Code's first version) (Revised RPS2000, 2008).
Table 2.1 - Summary of past, on-going and envisaged new activities

<table>
<thead>
<tr>
<th>Tasks already carried out with available results</th>
<th>Tasks to be developed within the project</th>
</tr>
</thead>
</table>
| **1- Inspection**  
Features surveyed and technologies utilized.  
Geometric survey available.  
Detailed inspection already carried out including materials, morphology.  
Fired earth bricks properties identified based on samples | The building is available for the application of additional inspection technologies resulting from the project. |
| **2-Structural Modeling and model updating** | To be developed |
| **3-Seismic analysis**  
Ongoing | To be improved using the new calculation criteria and methods derived from the project. |
| **4-Design of intervention**  
Restoration project | To be developed |
| **5- Implementing of intervention**  
Recent Restoration intervention | The strengthening could be executed during the project duration, and according to the methodologies resulting from the project. However, the possibility of implementing the proposed strengthening depends on availability of funds from local public institutions. |
| **6- Monitoring during and after strengthening implementation** | Available for monitoring during and after execution of strengthening. The implementation of a long term dynamic monitoring, as part of the activities of the project, is already envisaged. |
2.3 THE INTERNATIONAL CONSERVATION CENTER, ACRE, ISRAEL

2.3.1 Name, location and description

The international conservation center is situated on the southern edge of the old city of Acre next to the city wall in a building that offers remarkable views overlooking the bay of Haifa. It is a large two stories residence in an area of about 1000 square meters that was built during the end of the nineteenth century. On the ground floor however, the massive columns supporting the impressive vaults may suggest that part of the structure could even be from the crusader period, when this area belonged to the Pizanian Quarter.

The upper floor, which housed one of the wealthy families of Acre is rich with architectural elements typical to the "Lebanese houses" i.e. center hall plan where the rooms open to a spacious lounge, decorated with marble floors and pillars and decorative wooden ceilings. In the last decades the building hosted a youth hostel and was later deserted. Neglect and improper use damaged the structure and its special features. Acknowledging the responsibility we took with the declaration of Acre as a "World Heritage City" by UNESCO on the whole and the unique architectural and historical value of this house in particular, it is our duty today to restore it to its former grandeur, and revive its important role in the old city of Acre.

2.3.2 Historical note

2.3.2.1 The history of Acre

The city of Acre or Akko is one of the oldest continuously inhabited cities in the world, dating back to the time of the Pharaoh Thutmos III (1504-1450 B.C.E). It is a city of intrigue, where you can walk through labyrinthine alleys and streets and explore the remnants of Crusader, Muslim and Ottoman conquerors. Acre was once a leading port in the Middle East, in the same league as Alexandria and Constantinople. Today, it is home primarily to small fishing boats. A part of the Kingdom of Israel, Acre was incorporated into the empire of Alexander the Great after his conquest in 332 B.C.E. The city was subsequently seized by the...
Egyptian king Ptolemy II, who renamed the city Ptolemais in the 2nd century B.C.E. This name stuck until the Muslim conquest in the 7th century AD, when its ancient name was restored. Confusion over what to call the city was compounded by the Crusaders' conquest in 1104, after which it became known as St. Jean d'Acre, or Acre for short. In 1291, the Mamluks invaded and destroyed the city, killing every remaining Crusader and putting an end to the Latin Kingdom. Acre ceased to be a major city for almost 500 years. When the Bedouin sheikh Daher el-Omar carved a small fiefdom out of the Ottoman Empire in the mid-18th century, he made Acre his capital and built a large fortress. It was subsequently fortified by the Turkish governor (1775-1804), Ahmad Pasha al-Jazzer ("The Butcher"). The mosque al-Jazzer built is one of the most beautiful in Israel and the most distinctive building in the old city.

Napoleon landed in Palestine and assaulted Acre in 1799, but he was unable to take the city. His Middle Eastern campaign subsequently collapsed and he withdrew to France. Acre fell under Ottoman control until the Turks were defeated in 1918 by the British. The city subsequently became part of the British Mandate for Palestine. The British used the ancient fortress, which had never been breached, as a high-security prison to hold (and execute) members of the various Jewish underground groups. On May 4, 1947, members of the Irgun staged a dramatic rescue (dramatized in the film Exodus). Though few Jews escaped, the audacity of the raid was a serious blow to British prestige and a tremendous boost for the morale of the Jews. Today, the fort is the site of the Underground Prisoners Memorial Museum, which depicts the history of Acre and the prison. You can go into the death cell where the condemned were kept and the gallows where a noose still hangs above an open trap door.

On May 17, 1948, shortly after the Arab invasion, Israeli troops took control of Acre and most of the Arab inhabitants fled. It was subsequently incorporated into Israel after the War of Independence.

Check out Khan El Umdan, the 18th century structure where camel caravans once brought grain and produce from Galilee to the market. The name means "inn of the pillars," for the fine granite Herodian pillars brought from Caesarea to support the structure. The clock tower is a much later addition, built in 1906 in honor of the Turkish sultan Abdul Hamid.

Across from the mosque is the entrance to the subterranean Crusader city. You can walk down to different levels and see how the Turks built on top of the old city. One of the more spectacular rooms is the Knights' Halls, which the Hospitallers, the Order of the Knights of St. John, used as a fortress more than 700 years ago. Today, the main hall is used for concerts. The lowest level is the Crypt, a great hall that may have been used for great ceremonies by the Crusaders.

Elsewhere in the city is the home of Baha Allah, the prophet of the Baha'i faith. His tomb is in a park just outside of town along the Acre-Nahariya road. As in the headquarters in Haifa, the grounds have spectacular gardens.

Today, the population of Acre is approximately 40,000, but only 6,000 live in the old city. It has one of the higher proportions of non-Jews of any of Israel's cities, with roughly 25 percent Christians, Muslims, Druze and Baha'is. The city is a magnet for tourists and the home of the country's steel industry.

2.3.2.2 The construction stages of the international conservation center

After stage analysis, it appears that the building has at least 6 main construction stages:

Stage 1: this is the earliest stage, probably dating from the Crusader period, which includes massive, irregular remains at the north-western part of the building, a well and a water reservoir. The connection between the remains is unclear.

Stage 2: most of the construction at the entrance level is attributed to a later period, perhaps the Crusader/Mameluke period. The site contains at least two structures, while room 110 and the space connected to it formed a separate unit and the rest of the building might have functioned as one unit around a yard. The ground floor structure was vaulted, and probably didn't have a second floor (or it was destroyed).
Stage 3: an additional floor was built, and the structure became a two-floor yard house, perhaps at the beginning of the Ottoman period. A staircase to the second floor was added. The second floor overlooked the main yard, over a circular balcony. A barrel vault was built above the entrance to the yard, and acted as a wide "bridge" between floors.

Stage 4: 19th century - rich effendies rehabilitated the structure in the 19th century. The yard on the ground floor was covered, and the first floor, which nowadays is the main space, was built. Seams and construction technology in the ceiling, floor and eastern wall show that the yard area was decreased on its western size and covered. The barrel vault was enlarged, in order to support the main marble pillars" row (on the first floor). An arch was built to the south of the stairs, supporting a wooden ceiling covering the bottom of the stairs and supporting the northern marble pillars. Another wooden ceiling, with heavy crosswise beams, was built between the enlarged vault and the arch. The upper space was covered by a roof with a decorative ceiling and paved with Carrera marble. When construction was completed, all the rooms on the first floor faced a new, magnificent large space that covered the yard and had a view to the street and the sea to the south, through a Trifora window. The stone steps were enlarged by half a meter (probably by blocking the hamam's ventilation opening) and covered with marble.

Figure 2.23 second stage - yard house

Figure 2.24: third stage - a two-floor yard house
Stage 5: British mandate - the building was divided between several families, who lived in small units in different parts of the building.

Stage 6: 1960's and 1970's - the building was used as a hotel, and later as a youth hostel. The building's area was enlarged by annexation of the eastern wing of the upper floor and of the large hall on the ground floor.

2.3.3 Historical and artistic significance

The building has a high cultural, historical and urban value. It was used as a governmental building for part of the stay of the Bahaullah in town. For the Baha’i community, this building has cultural significance. From an esthetic point of view, the building is full of original architectural elements typical to Acre, such as marble floors, painted ceilings, trifora windows and special stone carvings. Its main importance is in being a typical Acre "well-off" building, with sensitive inner construction. Its location in the Pisan neighborhood increases its importance in the city’s urban landscape.

2.3.4 Structural and material features

The international conservation center is a two-story building (approximately 15m tall). The ground floor has two wings, and the upper floor consists of a central building encircling a closed, covered yard. The upper floor has a main space surrounded by various rooms. Both the ground floor and the upper floor have additional internal galleries, which divide the space in two.

The basement floor consists of a barrel vault, upon which are supported the inner walls of the two one-story buildings constructed in stage 2. The original floor was close to water-level.

The ground floor is mostly covered by cross vaults and barrel vaults, but some parts are covered by wooden ceilings. The upper floor was originally covered by flat wooden ceilings and shingles, but parts were replaced by flat concrete roofs.
The building typology of the conservation center in Acre is similar to many buildings in ancient Acre. The gravel walls are significantly different between both floors: the ground floor has 80cm thick walls, while the upper floor has relatively thin, 40cm walls. The upper floor has a main hall, whose ceiling is supported by a system of gravel arches supported by a series of pillars. The pillars are supported on the wooden ribs of the main hall's enclosure, which is problematic (see further on).

The exterior southern wall of the upper floor's main space consists of a system of large windows, which are separated by marble pillars. This wall was built after the demolition of the outer balcony. The original building wasn't linked to adjacent buildings, but various additions on both floors and construction nearby have created links on the eastern and southern parts.

The ground floor's walls have differing widths, according to their role: pillars of different sizes (up to 180cm) form the base of the vaults, and outer walls ranging from 140-150cm and from 70-90cm where the ceiling is made of wooden beams.
The ground floor’s ceilings have different building types, according to their role, their period of construction and the load bearing method: as an arch, as a vault or as a flat ceiling, made of wood or concrete. The ground floor’s middle ceiling is partly built as cross vaults, partly by barrel vaults/wide arches and partly by a wooden floor. The main entrance hall consists in part of a large arch, connected to the southern wall, which was probably used as an entrance arch before construction of the upper floor.

The same main space has a middle floor made of beams and wooden ribs, which connect to a barrel vault/arch, and forms with them the floor of the main hall of the upper floor. The ceiling is made of two massive 30/30cm, wooden beams, which cross from east to west between walls and are separated by wooden secondary beams (ribs) of 7/14cm every 40cm. There is gravel and covering filling above the beams, and the upper side is made of marble floor boards, and the upper side is made of marble floor boards.
The rooms' ceilings were constructed during different periods and using different methods. There are ceilings built from a main wooden beam and wooden ribs. In some cases there is no main beam, and the ribs lie directly on the peripheral walls. Above the ribs there are horizontal wooden boards, and above them there is a filling of lime, sand and gravel. Some of the main wooden beams are supported by panel arches.

The rooms' ceilings are wooden ceilings made of wooden ribs, but the outer part is covered by a shingle roof. The ceiling is built as a flat ceiling, without the lime, sand and gravel filling above the ribs' horizontal wooden boards. Nowadays the space above the ribs is empty. As noted, the lower part has decorated ceilings or wooden ribs.
Concrete ceiling roofs appear on some of the upper floor ceilings, which act as a roof. They were cast over the rib ceilings, which were used as a form, or after dismantlement of the wooden ceilings. In some cases they are not cast at the original height of the rooms. In a few cases a concrete beam was cast on the walls, and above it a concrete ceiling. The ceilings are 20-30cm thick, and most of them are bare on their vertical part.

There is also a support system, made from a series of 3 half-arches and a barrel vault, which creates horizontal support between the existing structure and the nearby western structure.
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Figure 2.34 - Horizontal support from the conservation center to the attached building.

Figure 2.35 - Ground floor plans.
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Figure 2.36 - First floor plans.

Figure 2.37 - Gallery floor plans.
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Figure 2.38 - Roof plans.

Figure 2.39 - Cut 1-1.
2.3.5 Present and foreseen future use. People at risk.

During the 1960’s and 1970’s the building was used as a hotel and later on as a youth hostel. Starting from the 1990’s, the center was used as an open school for conservation research. A few months ago “the International City of Rome Conservation Center” was inaugurated. The conservation center will be activated by the Antiquities Authority, the Society for Development of Acre and The Acre municipality, and will be supported by the Italian government. The center will address all audiences - the general public, and specifically students, and will also offer professional academic courses and international collaborations. The center will collaborate with various universities in Israel, in Italy and in other places, specializing in conservation. Additionally, the founders of the International City of Rome Conservation Center aim to train the residents of Acre in conservation: all international courses include Acre residents, thanks to a special system of scholarships awarded by various sources.

In the future a professional laboratory will be built in the center, which will allow conducting research on conservation agents, using Italian knowledge, experience and support.

2.3.6 Considerations on valuable cultural contents

Note that the conservation center building does not contain cultural art such as paintings, objects etc. However, many construction and decorative elements incorporated in the building recount of its function, the people using it and mostly, of its past.

1. While renovating the building in 2009, a complex of underground chambers was uncovered. The well structure had a marble pillar base, with a hollow center. Its upper part had 4 vertical holes and the inner diameter contained wider holes, facing each other. The well has a rectangular cut, and it is approximately 5 meters deep. The well’s side has alcoves, allowing climbing and descending into the well.
2. The marble floor on the first floor is made of pale, thin marble, with dark terrazzo cast grouts. The floor is rectangular and has a black circumference line, and edges without grouts. The grout junctions are usually pale marble cubes, which have been replaced in some places by a modern paving fragment.

Figure 2.41 - The western basin is covered and separated from the main space.

Figure 2.42 - The western basin is covered and separated from the main space.

Figure 2.43 - View of the marble floor.
3. The marble pillars are wider towards their bottom and have carved tops and marble cornices (some of the cornices in the room are painted). Beneath the marble cornices, there is a marble coating on a stone that supports the pillar. The top has a rotating flower pattern, and a concave upper cranny.
4. Decorated staircase. The banister is anchored to the stairs by the marble coating.

![Decorated staircase](image)

Figure 2.47 - Decorated staircase.

5. A chandelier including golden cedar tree engravings, with a flat upper surface. Some of the ornaments are missing. Above the ornaments are the remains of transparent glass, which was broken. The chandelier is not connected to the ceiling.

![Chandelier with wooden ornaments](image)

Figure 2.48 - A chandelier with wooden ornaments.

6. Wooden ornaments along the decorated ceiling in the rooms. The wooden ornaments surrounding the zinc tin include 5 different cedar tree ornaments.
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Grant Agreement n° 244123

Knowledge based assessment vaults

Figure 2.49 - Wooden ornaments along the decorated ceiling.

7. Decorated wooden ceiling with details of lamp bases in the beams.

Figure 2.50 - Decorated wooden ceiling.

Figure 2.51 - Details of wooden ornaments.

8. A colourful painting was discovered beneath the white paint, which requires uncovering and conservation. Following this discovery, wooden rods were discovered behind the zinc tin.
2.3.7 Present condition and damage

2.3.7.1 Considerations on the present condition
The building does not suffer from serious, constructive-geometrical stability problems. There are certain defects and other constructive and geometrical problems in various points, which require thorough treatment. There are no depressions in the vaults, which would have presented a constructive geometrical hazard. There is deviation of the walls from the horizontal plane on the upper floor, near the inner thickening of the south-western room on the upper floor. There are vertical cracks along the building, all along the south-western wing. The partial rehabilitation conducted in 2009 included the repair of a very problematic point, of geometrical defects on the upper floor’s floor in the parts where it is made of wooden ribs, above the entrance hall. Moreover, geometrical defects of wood bundles on the roofs, which suffered from microbiological damages, cracking and depression, including entrance of humidity to the walls and rooms, were repaired.

2.3.7.2 Main observed damage and decay
The main problem with the floor of the main hall on the first floor was that all 3 30/30cm wooden beams were eaten by wood bugs and there edges were rotted by water. The walls' support points were only 5cm.
Another major problem is the marble pillars placed on the upper floor's floor, which support the decorated ceiling and the shingle roof above it. The pillars rest on the ceiling's wooden ribs, and not directly on the ceiling.

Vertical cracks:
Appear along the outer southern wall. A test performed on site proved that the crack runs along the entire joint.

Defects in connections between walls and floors:
It can be seen that there are protruding areas, pointing to a defect in the horizontal connection of the middle floor and the connection of the upper floor's roof to the outer walls, such as the north-western front.
corner dismantlement:
Anchoring of iron parts, beams, and sealing of the draining system in the outer walls have created cracks, and later on caused the dismantlement of stone parts, mainly in the buildings corners.

Lack of stone in the walls and crumbling of the filling material between joints:
In many un-concentrated areas it is possible to see a lack of stones constructing the front. This lack is due to rock dismantlement, due to connected elements or due to increased erosion. Outer plaster sections and joints between stones in the inner walls connected to the outer southern walls are crumbling in their top and bottom parts due to penetration of rain water from the roof and from the front. Additionally, the location of large windows at the front causes rapid evaporation cycles, which encourage wall crumbling. In other parts of the building the plaster is crumbling and the joints do not contain covering material.

Addition of cement mixture:
In areas where cement lintels have been installed, the cement is crumbling, the metal is rusted and/or crumbling, and the beam and gravel wall surrounding it are bloated. This problem is visible mainly on the upper floor, in the western and northern front and in rooms with direct access to the roof.

Figure 2.60 Addition of cement mixture

Geometrical state of the reinforced concrete roofs:
Most of the roof is covered by separate concrete roofs, based on the space’s height and area inside the building. The concrete walls were, of course, cast after concrete started being used in Acre, but most of them were probably cast after 1960. Most of them are in rough geometrical-physical condition: in some cases the concrete is crumbling, with large spaces between the casting and the gravel wall. Inner reinforcements visible from the roof’s circumference are rusted, bloated and/or crumbling. In some cases the concrete roofs are elevated at their edge or in places where the connection to the wall is weak. There are places where the tar layer separating the wooden roof from the concrete roof has not been removed, and a slit separates the two types of roofs. The place is exposed to rain-water and humidity. Additionally, there are IP iron beams which were used as reinforcements, which are twisted and extremely rusted. The main part of the roof has cracked and depressed wooden beams, but they are covered by reinforced concrete layers and the shape of the wood cannot be examined.

Figure 2.61 - Most of the concrete roofs are in bad geometrical-physical shape.  
Figure 2.62 - Crumbling beams and rusty exposed reinforcements.

State of the shingle roofs: 
In an examination of the roof conducted on February 2010, it was found that the wooden system bearing the roof is rotted in many areas. Penetration of rain-water and many years of negligence have accelerated the erosion process.
2.3.8 Information on local seismicity

Because Israel is located at the juncture of two tectonic plates, known as the Great Rift Valley, it is hazardous. Following is a partial list of strong earthquakes in the region:

Table 2.2 - Past earthquakes.

<table>
<thead>
<tr>
<th>When</th>
<th>Earthquake intensity</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>747 AD</td>
<td>Unknown</td>
<td>Presumed origin: the Jordan Valley. Known as “the seventh earthquake”. Beit Shean and Hamat Gader were destroyed. Extensive damages in Sousita, Tiberia and Jericho. Damages in Jerusalem.</td>
</tr>
<tr>
<td>1033</td>
<td>Unknown</td>
<td>Presumed origin: the Jordan Valley. Jericho was destroyed, as well as large parts of Ramle and Nablus. A tsunami hit Acre.</td>
</tr>
<tr>
<td>1068</td>
<td>Unknown</td>
<td>Presumed origin: Eilat Bay or the Arava. More than 15,000 killed. Extensive damages throughout the country, mainly in Ramle and its surroundings. People killed and damages in Eilat, the Banias and Jerusalem</td>
</tr>
<tr>
<td>1202</td>
<td>Estimated above 7.0 on the Richter scale</td>
<td>Originated in the Galilee Panhandle - Lebanon Valley. More than 30,000 killed and extensive damages in all inhabited areas of the Levant (Syria, Lebanon, Israel)</td>
</tr>
<tr>
<td>1759</td>
<td>Estimated above 6.5 on the Richter scale</td>
<td>Presumed origin: the Lebanon Valley. More than 3,000 killed. Settlements in Galilee, in Lebanon and around Damascus were damaged. Tsunami waves of 2.5 meters hit Acre.</td>
</tr>
<tr>
<td></td>
<td>Between 6.5 to 7.0 on the Richter scale</td>
<td>Thousands killed. Safed was completely destroyed, enormous damages in Tiberia.</td>
</tr>
</tbody>
</table>
The national territory is subdivided into seismic zones, depending on the local hazard. The hazard is described by the value of the reference peak ground acceleration on type A ground, agR. In the Israeli Standard (SI 413:2004) the expected ground acceleration is a forecast of the peak of the horizontal ground acceleration due to an earthquake, agR, for which there is a 10% probability that a stronger acceleration will occur at least once within a period of 50 years.

Table 2.3 Ground acceleration for Acre

<table>
<thead>
<tr>
<th></th>
<th>Ground acceleration</th>
<th>Geographic coordinates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>agR</td>
<td>Longitude</td>
</tr>
<tr>
<td>Acre</td>
<td>0.17</td>
<td>35.08</td>
</tr>
</tbody>
</table>

Figure 2.64 - Seismic zones of Israel.
2.3.9 Previous and on-going studies

Since the building was purchased by the Acre Development Company several years ago, and later by the Antiquities Authority, there is no information concerning researches conducted on the conservation center building. The structure contains testimonies of various changes, but it does not appear that those were made based on feasibility and examinations of the building.

2.3.9.1 Inspection works

In the last years, the Antiquities Authority conducted data collections and surveys, as detailed below:

- Measurements done on buildings meant for conservation.
- Preparation of a documentation file for the building, detailing its statutory state, its historic and cultural background, detailed analysis of stages and documentation of architectural details.

2.3.9.1.1 Geometric survey

The geometric survey examined the various methods of construction of the conservation center, diagnosed the constructive and physical condition of all its components and identified its defects and their causes. Additionally, a photogrammetric survey of all different parts of the building was conducted.

2.3.9.1.2 Research on materials

Samples were taken from main beams in the middle ceiling in order to identify the wood type and check its physical condition. The wood was identified as cedar, and a 30-40mm layer in the beam’s circumference was damaged by termites. Hence, not all wood slices can be used for the statistical calculation.

- The mortar and plaster materials used in the building are:
  - Earth and hot lime
  - Quicklime and beach sand
  - Quicklime, beach sand and fibers.

Following are the results of lab tests performed on similar historical buildings in ancient Acre:
<table>
<thead>
<tr>
<th>N.</th>
<th>Collecting area</th>
<th>Binder</th>
<th>Lime crumbs</th>
<th>Aggregate</th>
<th>Addition</th>
<th>Size of the clasts (mm)</th>
<th>Relation A/B</th>
<th>Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Crusader stronghold, money tower, mortar on the roof with canalier</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>coal</td>
<td>0.05-4 quartz 0.1-0.25 carbonates 0.4-4</td>
<td>1.5</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>Crusader stronghold, money tower, stair, upper floor. Ottoman plaster in two layers.</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>sand 100%</td>
<td>-</td>
<td>0.1-0.2</td>
<td>&lt;1.5</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>Crusader stronghold, money tower, indoor, first floor, stalactite, Ottoman plaster.</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>sand 100%</td>
<td>-</td>
<td>0.05-0.4</td>
<td>&lt;1.5</td>
<td>15</td>
</tr>
<tr>
<td>5a</td>
<td>Crusader stronghold, outdoor water pipe on the stair of the court, crusader plaster, upper layer.</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>coal</td>
<td>0.05-0.7 quartz 0.1-0.15 carbonates 0.25-0.4</td>
<td>1.5</td>
<td>30</td>
</tr>
<tr>
<td>5b</td>
<td>Crusader stronghold, outdoor water pipe on the stair of the court, crusader plaster, inferior layer.</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100% (earthenware)</td>
<td>(coal)</td>
<td>0.05-1.8 +0.5-0.7</td>
<td>1:1</td>
<td>15-20</td>
</tr>
<tr>
<td>6</td>
<td>Crusader stronghold, latrine, mortar in the bathroom.</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>earth (%), (coal)</td>
<td>0.05-1 quartz 0.07-0.17 carbonates 0.15-0.85</td>
<td>2:1</td>
<td>20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>N.</th>
<th>Collecting area</th>
<th>Binder</th>
<th>Lime crumbs</th>
<th>Aggregate</th>
<th>Addition</th>
<th>Size of the clasts (mm)</th>
<th>Relation A/B</th>
<th>Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Crusader stronghold, latrine, mortar of the masonry</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 85% earthenware 15%</td>
<td>-</td>
<td>0.05-0.6 quartz 0.1-0.2 earthenware 0.3-0.4</td>
<td>1.4</td>
<td>15</td>
</tr>
<tr>
<td>8a</td>
<td>Crusader stronghold, indoor, under the Columns room, crusader plaster of the reservoir, upper layer.</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>earthenware 95% sand 5%</td>
<td>(coal)</td>
<td>0.05-3 +0.3-0.6</td>
<td>1:2</td>
<td>20</td>
</tr>
<tr>
<td>8b</td>
<td>Crusader stronghold, indoor, under the Columns room, crusader plaster of the reservoir, lower layer.</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>(coal)</td>
<td>0.1-0.17</td>
<td>1.5</td>
<td>20-30</td>
</tr>
<tr>
<td>11</td>
<td>Crusader stronghold, prison, indoor vaults, crusader h=300cm from stamping level.</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>sand 90% earthenware 10%</td>
<td>(coal)</td>
<td>0.05-1 +0.15-0.25</td>
<td>1:5</td>
<td>20-25</td>
</tr>
<tr>
<td>12</td>
<td>Crusader stronghold, under the tower, Saint John church, under the Turkish bath, original crusader mortar.</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>marble 85% sand 45%</td>
<td>-</td>
<td>0.03-0.85 +0.25-0.5 quartz 0.15-0.2</td>
<td>3:1</td>
<td>20</td>
</tr>
<tr>
<td>13</td>
<td>Crusader stronghold, under the stair, Saint John church, room 1 h=160cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 60% earthenware 40%</td>
<td>coal</td>
<td>0.05-1.1 quartz 0.1-0.2 earthenware 0.3-0.5</td>
<td>1:4</td>
<td>20-30</td>
</tr>
<tr>
<td>17</td>
<td>Crusader stronghold, southern crusader street (graffiti), crusader plaster.</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>-</td>
<td>0.05-1.2 +0.1-0.25</td>
<td>1:2</td>
<td>20</td>
</tr>
<tr>
<td>N.</td>
<td>Collecting area</td>
<td>Binder</td>
<td>Lime crumbs</td>
<td>Aggregate</td>
<td>Addition</td>
<td>Size of the clasts (mm)</td>
<td>Relation A/B</td>
<td>Porosity (%)</td>
</tr>
<tr>
<td>------</td>
<td>---------------------------------------------------------------------------------</td>
<td>---------------------------------</td>
<td>------------------------------</td>
<td>-----------</td>
<td>---------------------------</td>
<td>-------------------------</td>
<td>--------------</td>
<td>--------------</td>
</tr>
<tr>
<td>17b</td>
<td>Crusader stronghold, southern crusader street (graffiti), mator of the wall, gratiflos</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>sand 100% (earthensware)</td>
<td>coal</td>
<td>0.1-0.8 +0.15-0.3</td>
<td>2:1</td>
<td>20</td>
</tr>
<tr>
<td>18</td>
<td>Crusader stronghold, fire room h=70cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>marl 55%  sand 45%</td>
<td>-</td>
<td>0.01-0.1 +0.15-0.2</td>
<td>2:1</td>
<td>20</td>
</tr>
<tr>
<td>19</td>
<td>Crusader stronghold, fire room, indoor vaults, crusader plaster</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100% (earthensware)</td>
<td>-</td>
<td>0.05-0.9 +0.10-0.2</td>
<td>1:2</td>
<td>30</td>
</tr>
<tr>
<td>20</td>
<td>Crusader stronghold, kotam room, h=150cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>-</td>
<td>0.05-2 quartz 0.1-0.2</td>
<td>1:5</td>
<td>15</td>
</tr>
<tr>
<td>21</td>
<td>Crusader stronghold, fire ante-room, indoor vaults, crusader plaster h=220cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>-</td>
<td>0.05-2.8 +0.10-0.2</td>
<td>1:5</td>
<td>15</td>
</tr>
<tr>
<td>22</td>
<td>Crusader stronghold, watchtower, indoor vaults, Ottoman plaster, h=220cm from the stamping level</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>earthensware 50% sand 50%</td>
<td>straw</td>
<td>0.04-1 +0.15-0.5</td>
<td>1:2</td>
<td>20</td>
</tr>
<tr>
<td>23</td>
<td>Crusader stronghold, Jodokovski Tower, indoor vaults, Ottoman plaster, h=200cm from the stamping level</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>earthensware 90%</td>
<td>-</td>
<td>0.05-0.1 +0.10-0.15</td>
<td>1:2</td>
<td>15-15</td>
</tr>
<tr>
<td>24</td>
<td>Tower with painted ceilings, indoor, Ottoman plaster, h=220cm from the stamping level</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>(straw)</td>
<td>0.05-0.2 +0.10-0.2</td>
<td>1:2</td>
<td>15-15</td>
</tr>
<tr>
<td></td>
<td>Town Sukh house with painted ceilings, indoor, Ottoman plaster, h=40cm from the stamping level</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 75%</td>
<td>(straw)</td>
<td>0.05-0.4 +0.10-0.2</td>
<td>1:3</td>
<td>15-30</td>
</tr>
<tr>
<td></td>
<td>Town, Sukh house with painted ceilings, indoor, Ottoman plaster, h=40cm from the stamping level</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 100%</td>
<td>(coal)</td>
<td>0.05-1.5 +0.15-0.2</td>
<td>3:1</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Town, Vito house, indoor, near the motioned window with three lights, Ottoman plaster, h=130cm from the stamping level</td>
<td>Lime or weakly hydraulic lime</td>
<td>no</td>
<td>sand 100%</td>
<td>(straw)</td>
<td>0.15-0.4</td>
<td>1:5</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Town, Genesee street, (Greek patriarchal) crusader second level, crusader mortar, h=100cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 85%</td>
<td>(coal)</td>
<td>0.05-1.5 +0.15-0.2</td>
<td>1:2</td>
<td>20-25</td>
</tr>
<tr>
<td></td>
<td>Town, private room 1162, court, Ottoman plaster in two trees</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 85%</td>
<td>(coal)</td>
<td>0.05-1.5 +0.15-0.2</td>
<td>1:4</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>Town, Templar tunnel before the forl, crusader mortar, h=200cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 95%</td>
<td>(coal)</td>
<td>0.05-1.2 quartz 0.1-0.15</td>
<td>1:2</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Town, Templar tunnel, right fork, crusader mortar inside the hole, h=210cm</td>
<td>Lime or weakly hydraulic lime</td>
<td>yes</td>
<td>sand 95%</td>
<td>(coal)</td>
<td>0.05-1.2 quartz 0.1-0.15</td>
<td>1:2</td>
<td>15</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
<td>Weakly hydraulic lime</td>
<td>sand 75%</td>
<td>(coal)</td>
<td>0.05-1.2 quartz 0.1-0.15</td>
<td>1:2</td>
<td>15</td>
</tr>
</tbody>
</table>

### 2.3.9.1.3 Visual inspection

A detailed visual inspection of various parts of the building, such as bearing walls, partitions, arches, vaults, wooden ceilings and building covering was conducted as part of the geometric survey. Maps were prepared for the morphological identification of materials in the building, but also for marking of the constructive and physical problems, such as cracks, crumbling of covering materials, depression of elements, wood rotting etc.
2.3.9.1.4 Deep inspection by means of NDT and MDT
Examination of the wooden beams covering the roof and supporting the middle floor’s floor was done by a visual inspection, knocking on the wood and manual penetration of up to 1cm for sample testing. A FLAT-JACK test was performed on similar buildings. No drilling or mechanical examination of humidity percentages was conducted.

2.3.9.2 Structural analysis
Statistical analysis of the main wooden beams on the first floor’s floor showed that the beams’ cut is not sufficient for load bearing according to the Israeli standard for areas meant for public gatherings. The wooden beams were rotten and didn’t have appropriate support points. Hence, the wooden beams were removed. A statistical calculation was performed for new wooden beams’ cuts and the floor was reconstructed according to the standard.

2.3.10 Previous or on-going restoration works
2.3.10.1 General restoration works
The center’s restoration works included several actions preparing the main spaces for use, but no thorough treatment of the building was performed. The main actions that were performed:

- Stabilization of cracks on outer façades.
- Replacement of stones where they were lacking or eroded.
- Replacement of shingles for roof sealing. The wooden roof hasn’t been stabilized yet in places where the wood is rotten.

Figure 2.65 - Removing shingles from the building's roof.

- Restoration of the wooden floor above the main entrance hall.
Figure 2.66 - Restoration of the wooden gallery.

- Dismantlement of concrete additions and removal of cement plaster from walls, arches, and stone vaults.
- Joint filling and applying of the stone façade.

Figure 2.67 Joint filling and applying of stone façades.

- Northern arch stabilization by joint filling and sealing of their upper parts.
- Supporting the upper floor's inner pillars directly on the existing stone arch and not on the original wooden ribs.
2.3.10.2 Actions oriented to seismic retrofitting

Even though no earthquake analysis was performed, a system of anchors and pulleys was installed on two floors on the western part of the building during the rehabilitation and conservation works. The calculations of the levers and pulleys system are good only for statistical analysis, but can be changed based on the earthquake calculations' results. Additionally, the half arches used for horizontal support to the adjacent building were restored.

2.3.11 Historical research

The Conservation Department of the Antiquities Authority conducted an historical research including detailed building stages, accompanied by historical pictures and various estimations for construction periods. There is no need for further historical research.

2.3.12 Local seismicity and characterization of seismic action

A study on the characterization of the seismic action for the conservation center wasn't already available. This information is considered necessary for the purpose of the project.
2.3.13 Proposed (additional) inspection works

2.3.13.1 Geometric survey: aims, methods and technologies
As mentioned, a complete photogrammetric survey is already available.

2.3.13.2 Research on materials: aims, methods and technologies
Non-destructive tests of different materials, such as covering materials and stone types, have to be conducted. These tests can date the building periods and provide characterization of the material, such as physical and mechanical properties.

2.3.13.3 Direct inspection.
Examination of the loads on main pillar heads in the upper floor and on the arches between them.

2.3.13.4 Deep inspection by means of NDT and MDT: aims, methods and technologies
Flat jack tests on all floors of the building, acoustic test and drilling of core mortar in walls

2.3.14 Proposed monitoring.

2.3.14.1 Static monitoring
The project will include installation of a monitoring system for cracks and depressions in problematic areas identified in the building. This monitoring can be useful, as cracks that were closed by the last geometrical conservation restoration have to be monitored for 3-5 years.

2.3.14.2 Dynamic monitoring
A dynamic system for building movement, especially since the building is located at the front of the port and on the city’s southern front.

2.3.14.3 Monitoring phases

2.3.14.3.1 Monitoring before intervention
The static and dynamic monitoring systems proposed will be working before the possible execution of the proposed seismic strengthening solution.

2.3.14.3.2 Monitoring and control during intervention
Both the static and the dynamic system will be active during the execution of the proposed seismic intervention.

2.3.14.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active for a period of two years after the implementation of the seismic strengthening.

2.3.15 Proposed structural analysis: aims and methods

2.3.15.1 Modeling and structural analysis methods
The modal response spectrum analysis is used, in order to assess the seismic behavior of the “typical” building of the Old Acre. Modeling and structural analysis in case of existing buildings and for seismic assessment purposes. It is possible to adopt the same analysis methodologies considered for the design of new structures:
- Lateral force analysis (linear static),
- Modal response spectrum analysis (linear dynamic),
- Non-linear static (pushover) analysis,
- Non-linear time history dynamic analysis.
2.3.15.2 Model updating and monitoring. Integration with monitoring
In order to receive a seismic model as close as possible to the problematic seismic conditions of Acre, a lot of effort has to be put into deciphering and developing dynamic monitoring data. The results from the numerical model can give an indication for sensor location in important areas in the building and for monitoring results.

2.3.15.3 Assessment of possible intervention methods
Dynamic monitoring, as was presented earlier, will be performed on the building. The proposed reinforcements will be added before execution.

2.3.16 Envisaged interventions

2.3.16.1 Considerations on the need for conservation, repair and maintenance
As mentioned earlier, lately some conservation and rehabilitation works were performed on the main wings, in order to be used as the conservation center. The works were funded by the Antiquities Authority, donations from abroad (Italy, the United States) and the Acre Development Society.

However, it is clear that the project needs to have a more specific contribution, from a scientific point of view, to understanding its static and dynamic stability, especially in light of the various additions. Checking the resilience of existing enforcements to earthquakes was performed lately. Formalization of propositions for building conservation and maintenance.

2.3.16.2 Considerations on the need for seismic strengthening and proposed actions
There is no doubt that this type of building in Acre has to be reinforced against earthquakes, since the outer walls are high relative to their thickness. Hence, during the last years the building was partially reinforced by a circumference pulley system that tied the walls. The research has to consider the efficiency of the strengthening systems and the necessity to improve their location and performance.

2.3.17 Potential contribution to the project

2.3.17.1 Compliance with project objectives
The conservation center building in Acre is typical of similar buildings in the Middle East. These types of buildings exist in places prone to earthquakes and rough climate conditions, and do not comply with earthquake standards or new standards. These cultural assets can cause loss of human life, as well as loss of important cultural assets, of major urban and national importance in the Middle East.

2.3.17.2 Foreseen opportunities for application of new approaches and technologies
Since some of the building is in bad shape and necessitates replacement of parts, such as wooden beams, this is an opportunity to examine the system connecting vertical and horizontal elements, and to use new technologies.

2.3.17.3 Foreseen opportunities for testing and validation of proposed intervention methods
Methods proposed after the application research can be used on the building (which hasn't been completely stabilized yet). There is also a possibility for long term monitoring.

2.3.17.4 Potential contribution to validate the project’s methods and criteria
Since the building is a public building belonging to the Antiquities Authority, the influence of the different methods and criteria can be tested on similar buildings in other historical coastal cities.

2.3.17.5 Possibility of monitoring long-term effectiveness of intervention
Since the building is a public building belonging to the Antiquities Authority, a long term, protected monitoring system can be installed.

### 2.3.17.6 Other specific opportunities provided by the proposed case study

Applying this project in a place used as a conservation center, which offers guidance to all conservation professionals, can be an excellent opportunity for presenting the research method and its results, at any time.

Table 2.4 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th></th>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1- Inspection</strong> (specify methods and technologies)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>Complete photogrammetric survey is already available</td>
<td>Physical properties of mortar and stone, and mechanical properties of stone have to be identified</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 1.3 Internal morphology | - Pulse radar and seismic tomography.  
- Dynamic monitoring  
- Temperature distribution monitoring.  
- Structural analysis by means of non linear FEM and model updating.  
- Dynamic survey by non contact radar interferometer. | |
| 1.4 Other | Foundation soil has not been characterized by geotechnical | |
| **2. Monitoring** | | |
| 2.1 Static monitoring | Not available | |
| 2.2. Dynamic monitoring | Not available | Dynamic monitoring system is required especially because the building is located in the front of the port and in the southern part of the city |
| **3. Structural analysis** | | |
| 3.1 Modeling and model updating | Not available | Activities to be developed within the project |
| 3.2 Seismic analysis | Not available | Activities to be developed within the project |
| **4. Intervention** | | |
| 4.1 Characterization of needs for repair, maintenance and (seismic) strengthening | Repair and maintenance needs already defined. | Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria. |
| 4.1 Design of strengthening intervention | The strengthening intervention was made only for static stability | Should be reconsidered and improved based on the project’s new methodologies and criteria for |
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<table>
<thead>
<tr>
<th>Knowledge based assessment vaults</th>
<th>seismic assessment. Possibility to apply incremental approach based on continuous monitoring</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.2 Real implementation</td>
<td>Strengthening proposal tentatively will be accepted by property</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td>The static and dynamic monitoring systems proposed will be working before the possible execution of the proposed seismic strengthening solution</td>
</tr>
</tbody>
</table>
2.4 MEKAAD RADWAN, CAIRO, EGYPT

2.4.1 Name, location and short description

Mekaad Radwan (Figure 2.70) is one of the civil architectural buildings that were founded by the ottoman pasha Radwan Al-Faqary who used to be the governor of Al Mansoura governorate in the ottoman period in Egypt. The Mekaad has been built in 17th century (1635 AD). The building is located in old Cairo front of Bawabet Elmetwalli and behind Souq El Khayameya. The monument is a part of a complex known as Radwan Palace or Qasabet Radwan which carries number 406 in the Egyptian classification of Islamic and Coptic monuments, while Mekaad Radwan carries number 208 which is a separate number apart from the main monument. The references available on the place and surrounding territory don't provide sufficient data to trace a historical sequence up to the present time. The all known data reveals to a high position of Radwan Pasha during the ottoman era, no written resources are available on that topic. The local data taken from the residences surround the monument mention that there was a high destruction after the 1992 earthquake in its walls and ceiling. Restoration efforts were achieved by a local construction company, but the optical investigation of these previous activities has detected many mistakes in the techniques and the materials used, for example Portland cement was used a plastering material and a main grout for injecting the faults.

![Figure 2.70 - (a) The Mekaad Radwan, and (b) Sketch map showes the location of Mekaad Radwan.](image)

2.4.2 Historical and artistic significance

Mekaad Radwan includes some valuable decorations from mosaic and decorated ceilings as described as following.

2.4.3 Architectural arrangement, structure and materials

Masonry and timber are the most common building materials that can be found widely used in Egyptian buildings. Naturally, innumerable variations of these materials, techniques and applications occurred during the course of time. The influencing factors were mainly the local culture and wealth (buildings in Lower Egypt are more rich than those of Upper Egypt), the knowledge of materials and tools, the availability of material and architectural reasons. In some buildings, traditional building materials were almost exclusively local materials, usually used without much processing or with a minor improvement using elementary tools. The external walls of the most ancient buildings are made of irregular masonry. The stones are of variable size,
sometimes with stone slabs used as an external ornament. Small stones and brick fragments are used to fill larger voids, with joints made of clay or clay-lime mortar. Walls are, usually, rendered with lime or clay mortar and painted with lime wash, creating flat and white facades suitable for applying colored scenes. Timber (or, rarely, stone) lintels strengthen window and door openings. The timber beams that sustain the floors and roofs are supported directly in cavities in the external masonry walls (which become later an aperture for rain water to the inside). The internal walls can be built with different techniques; these techniques are very clear in the western wall of the Mekaad Radwan. Then, the surface is plastered and painted with lime wash. The Mekaad Radwan building has a rectangle shape plan (13x6 meters) with the longitudinal sides approximately WNW-ESE. It reaches up to 15 meters above the street level and is consisting of three storage namely: the ground floor; the opened floor (The Mekaad) and the living floor (see Figure 2.71 to Figure 2.74).

Figure 2.71 - A detailed planning and the outdoor lines of the Architecture arrangement of Mekaad Radwan.

Figure 2.72 - 3D (three dimensional ) prospective of Mekaad Radwaan.

Figure 2.73 - 3D prospective of Mekaad Radwaan with focusing on the Mekaad floor and its relation to the upper and down part.

Figure 2.74 - The façade of Mekaad Radwaan building.
The building has only a gate which is introduced by stairs on the northern front side. The ground floor is occupied by large room that is taken up now as a workshop. The floor has three windows and the workshop door occupying the ground floor façade. This ground floor (Figure 2.75) is being divided into 2 stores with 5.5m and 3m height and 6m depth for each one; this part of the monument is the most deteriorated part as it suffers from the effect of ground water which covers its basement till approximately 75cm height. The second floor (the superstructure) is the main body of the Mekaad, this word stands for seat or chair or saloon in its exact meaning. This floor is the richest part in the monument as it has many of the architectural units including wooden ceiling, mosaics, and a 3 meters span decorated wooden balcony (with double pent-roof). This floor may be also called the “Mekaad Floor” where it is bounded by three wall sides while the fourth (the northern) side is opened. This floor is occupied by a large meeting room with an area of about 60m square with dimensions 6x10 meters. The room has an entrance within its western side wall, but now it is sealed adjacent to a mosaic covering half wall by red brick and plaster. The western wall has two wooden doors leading to shallow wardrobes inside the wall. The lower part of the eastern side wall is covered by marble mosaic while its upper half is covered by plasterless stone blocks. The southern longest side is occupied by recent restored plastered wall. The fourth opened (free) side of the Mekaad floor is mainly distinguished by two marble columns carrying three pointed arches (Figure 2.76 and Figure 2.77). The 3rd floor contains sleeping and leaving rooms, but it couldn't be accessed because it is being inhabited at the moment of local residences. A side separated iterance on the right hand with 3 m wide leads to the stairs for this housing or living floor whose façade is occupied by three large wooden windows (Figure 2.78).
### 2.4.4 Considerations on valuable cultural contents

#### 2.4.4.1 Mosaics

Islamic architecture used mosaic technique to decorate religious buildings and palaces since Muslims conquered the eastern provinces of the Byzantine Empire. In Syria and Egypt the Arabs were influenced by the great tradition of Roman and Architectural arrangement, structure and materials Early Christian mosaic art. During the reign of the Umayyad Dynasty mosaic making remained a flourishing art form in Islamic culture and it is continued in the art of (Azulejo) in various parts of the Arab world.

The first great religious building of Islam, the Dome of the Rock in Palestine, which was built between 688 and 692, was decorated with glass mosaics both inside and outside. Only parts of the interior decoration survived. The rich floral motives follow the Roman traditions, and "Islamic only in the sense that the vocabulary is syncretic and does not include representation of men or animals. Mosaics in Mekaad Radwan are a good example for these beliefs as they don't represent human beings or animals or even plants, the mosaics are composed of geotechnical shapes with different colors, both marble and glass were used in fabricating the mosaics in Mekaad Radwan (Figure 2.79 and Figure 2.80).

![Figure 2.79 - Marble mosaics of the eastern wall of Mekaad Radwaan room.](image)

![Figure 2.80 - Marble-glass mosaics of the eastern wall of Mekaad Radwaan room.](image)

#### 2.4.4.2 Decorated ceiling

The wooden ceiling of the second floor and the ceiling of the wooden balcony in the same floor of Mekaad Radwan are being decorated with polychrome pigments as shown in (Figure 2.81, Figure 2.82, and Figure 2.83). It's well-known that the decoration of ceilings in the Islamic architecture is done through one of two techniques. The first and most common technique is the outlining of the design and the shape of ornamentation followed directly by the application of the pigments on the paint ground. The second technique which is rarely used involves applying different color grades and several pigments followed by coloring the upper layer with the desirable color grade. The latter technique is more advanced since it shows the experience and skill of the artist.

The optical investigation of the derived samples confirmed that the artist used the multi-layer technique. Thus it can be concluded that the painting layers of the Mekaad Radwan second floor ceiling was composed of a wooden support upon which the plaster layer was composed of two layers (gesso grosso and gesso sotile), and a pigment layer carried out using the tempera technique. While the wooden ceiling of the balcony has been decorated using the second technique which involves applying different color grades and several pigments followed by coloring the upper layer with the desirable color grade. The optical investigation didn't reveal any plastering layers in the ceiling of the wooden balcony.
2.4.5 Present condition and damage

2.4.5.1 Main observed alterations, damage and decay
Geometric characteristics of the damages and deterioration phenomena affecting in Mekaad Radwan monument were determined in this section. The phenomena resulted from different causes including earthquakes and preexisting October 1992 seismicity event, are represented by extensive cracking, subsiding, corrosion and other failure phenomenon. Accurate field survey including description and measurements was carried out for the archaeological significant damaged parts of the monument particularly the mekaad meeting room.

The Mekaad Radwan building consists of three storey, namely in this study as the ground floor with its façade, the Mekaad floor with its western and eastern side walls and the roof floor which is not available for studies because it is currently used as housing place .The building has a wooden gate located within the western front side which form what is called the gate façade.

The building is made of thick load bearing walls and wooden beam floor structures on shallow foundation level. All windows and doors have typical wooden lintels which accomplished in some cases, the function of a tie beam.

The monument site was generally visually inspected and five categories of architectural elements were emphatically investigated as given in Table 2.5.

- The first category includes the ground floor façade where 8 damaged sits were investigated as shown in Figure 2.84 to Figure 2.88.
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The second category includes “the gate façade” where 17 damaged site were detected as shown in Figure 2.89 to Figure 2.94.
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Figure 2.89 - Significant evaluated separation and dislocation of stones, and mortar loss and cracking at the gate façade of the building.

Figure 2.90 - Extensive deformation deterioration and stone loss creating irregular holes and cracking in pier to the right of building gate.

Figure 2.91 - Shallow opened space and separation between stone block around the building gate.

Figure 2.92 - Evaluated separation between stone block in the corner adjacent to the building gate.

Figure 2.93 - Dangerous evaluated dislocation of the gate arch.

Figure 2.94 - Significant separation within frontone and irregular crack in the plaster underlying a window within the gate façade.

- The third category includes the "western side wall" inside the Mekaad room where 10 sites of damages were geometrically surveyed as shown in Figure 2.95 to Figure 2.99.
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Figure 2.95 - Mortar and plaster loss and irregular cracked plaster in the western side wall in the Mekaad room.

Figure 2.96 - Irregular bifurcate (Y-shape) cracked plaster.

Figure 2.97 - Vertical, slightly irregular cracked and plaster loss and separation between the wooden frames of door.

Figure 2.98 - Slightly irregular crack and plaster loss upon red brick.

Figure 2.99 - Oblique crack plaster inside the wardrobe in the Mekaad room.

- The fourth category includes the "eastern side wall" inside the Mekaad room where sites of damages within or around the mosaic wall were geometrically survived, as shown in Figure 2.100
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The fifth category include the ground room of Mekaad floor as well as its opened side with columns and arch where sites of damages were investigated as shown in Figure 2.101.

Figure 2.101: The ground and opened side of Mekaad room the fifth category showing the situation of tile, column and separation of balcony.

2.4.5.2 Effects of October - Cairo Earthquake
Several effects of the October - Cairo Earthquake can be observed in the study site, of which:

1) The timber lintels or beams that strengthen the window and door opening or sustain the floors and ceilings of the Mekaad Radwan and adjacent buildings, are completely collapsed (Figure 2.102) or fractured, bended and dislocated (Figure 2.103) due to over loading and possible differential settlement that may be caused by partially liquefaction of the foundation soil.

Figure 2.102 - The effect of the earthquake on the adjacent wall to the ground floor of building showing collapsed timber lintel and stone dislocation.

Figure 2.103 - Dislocation and rupturing of the timber lintel at the western wall as result of the earthquake.

2) Shear failure in the form of diagonal or oblique cracks (Figure 2.104) where the inertial forces are in the plane of the walls.

Figure 2.104 - The eastern side wall (the fourth category), the lower part of which is covered by mosaic that bordered by mortar loss. Note the oblique crack within the marble mosaics.
3) Nearly vertical cracks in the marbleized (Figure 2.105) due to shear forces creating from the earthquake.

4) Separation between the edge of the eastern wall and the wooden balcony due to the ground movement during the earthquake (Figure 2.106).

5) Rupturing and dislocation of many blocks with locally modifying the stress state distribution that caused in the ruin of a large portion of the ground floor façade particularly the workshop entrance (see Figure 2.85).

6) Displacement of the frontone due to the stresses creating from the possible soil settlements (see Figure 2.86 and Figure 2.88).

7) One of severe evolutionary cracks affect the gate façade of the building that are continently creating a parallel separation of the non-plastered stone blocks (Figure 2.107).

8) The slow developing dislocated of the stone blocks and the arch of the gate façade (Figure 2.108), due to increasing weakness of the structure and continuous increasing of the stress after the earthquake.

Figure 2.104 - Diagonal and vertical cracks in the adjacent wall as result of shear forces.

Figure 2.105 - Nearly vertical cracks in the marble mosaics as result of shear forces.

Figure 2.106 - Separation between the edge of the eastern wall and wooden balcony due to the ground movement.

Figure 2.107 - Evaluated dislocation and cracking as result of previous earthquake (notice the cracking of the gypsum spy).
Causative factors for plaster-mortar loss

The plaster and mortar of the Mekaad walls were subjected to loss and cracking (see Table 2.5). The reasons for loss of these materials could be categorized into internal and external reasons. The internal reason are concerning with petrographic characteristic of the building stone and bricks as well as the type and composition of mortar and plaster used. The external reasons may be relate to the construction deficiency, exterior salt invasion or differential settlement and subsidence as well as the earthquake affects. The invasion of the mortar or plaster by internal salts present in the stone or brick or by expansion of the gypsum plaster can be observed particularly in the western side wall (e.g. site No 31 & 32 are example) and the upper part of the gate façade (e.g. site No 20 & 23) which are completed by red brick. The possible salts in the bricks can be gradually released over a long period to the surface causing plaster loss. This process could be happened through the absorption of humidity from the air by the salts inside the façade followed by release of salts to be brought to the surface of the brick and to crystallize out by the evaporation of water holding them solution. The loss and cracking of the plaster may be also due to the formation of ettringite which is considered as internal sulfate attack. It occurs mainly in gypsum mortars and plasters which are exposed to high relative humidity. Damage to the gypsum occurs when ettrinite crystals exert an expansive force within the plaster as they grow. The expansion usually causes cracking and damage to the plaster or to mortar. Deformation of the plaster may be occurred as result of construction deficiency where surface plaster layer could be shrinked by rapid drying plastering process and due to the restraint provided by the plaster, resulting in shallow cracks of varying depth. Definitional settlement of the foundation soil as well as earthquake events may cause a variety of problems in the plaster layer, from cracking and failure.

Table 2.5 - Distribution and geometrical characteristic of the damaged sites and deterioration phenomena.

<table>
<thead>
<tr>
<th>Category site</th>
<th>Architectural description site</th>
<th>Site No</th>
<th>Damage material and damage patterns description of Dangerous evaluated</th>
<th>Geometrical dimensions (cm)</th>
<th>Possible causes of the damages</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) The ground floor façade</td>
<td>The ground floor façade is occupied by large room that is taken up now as workshop. The façade has there windows and work shop door.</td>
<td>1</td>
<td>deformation , dislocation of blocks</td>
<td>Length Width Depth</td>
<td>Extensive chemical by weathering and subsoil water Different settlement of foundation soil (creating subsidence) that enhanced by Earthquake</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>Leaning of stone block</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>Stone loss ,comities &amp;deformation</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>Significant dislocation of the stony lintel</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Description</td>
<td>Level</td>
<td>Probability</td>
<td>Annual Loss</td>
<td>Category</td>
</tr>
<tr>
<td>---</td>
<td>------------------------------------------------------------------------------</td>
<td>-------</td>
<td>--------------</td>
<td>-------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>5</td>
<td>Significant displacement of highly eroded frantone</td>
<td>22</td>
<td>1</td>
<td>0.5</td>
<td>Earthquake</td>
</tr>
<tr>
<td>6</td>
<td>Extensive stone erosion and stone and mortar loss</td>
<td></td>
<td></td>
<td></td>
<td>Weathering and mortar loss</td>
</tr>
<tr>
<td>7</td>
<td>Significant displacement of eroded frantone</td>
<td>22</td>
<td>3-4</td>
<td>1-3</td>
<td>Earthquake</td>
</tr>
<tr>
<td>8</td>
<td>Stone erosion &amp; cavity in stone block</td>
<td></td>
<td></td>
<td></td>
<td>Weathering</td>
</tr>
<tr>
<td></td>
<td><strong>(2) The gate façade of the building</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>The side gate's façade occupied by the main gate of building overlying by arch</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>and small squared window that up to the middle level of the level of the</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Makaad floor, overlying by large elongated wooden separated by lintels.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>The lower level of the roof floor is occupied also by an elongate window</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Extensive deformation deterioration and stone loss creating irregular holes</td>
<td>80</td>
<td>1-6</td>
<td>6-8</td>
<td>Karstification due to chemical</td>
</tr>
<tr>
<td></td>
<td>and cracking in pier the right of building gate</td>
<td></td>
<td></td>
<td></td>
<td>weathering by subsoil water</td>
</tr>
<tr>
<td>10</td>
<td>Dissolution and erosion of stone block face &amp; edges</td>
<td>90</td>
<td>20-30</td>
<td>8-9</td>
<td>and moisture effect that</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>enhanced by Earthquake</td>
</tr>
<tr>
<td>11</td>
<td>Shallow opened space between stone block</td>
<td>25</td>
<td>0.5</td>
<td>1</td>
<td>Loss of mortar in mortar joint</td>
</tr>
<tr>
<td>No.</td>
<td>Description</td>
<td>Code</td>
<td>Quality</td>
<td>Cause</td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>-------------------------------------------------------------------------------------------------------</td>
<td>------</td>
<td>---------</td>
<td>-----------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>Plaster loss from lintels underlying and overlying the middle window</td>
<td>-</td>
<td>-</td>
<td>Bad plaster &amp; enhanced by earthquake</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td></td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>Circular surface crack within the plaster underlying a window of roof floor</td>
<td>90</td>
<td>1</td>
<td>1</td>
<td>Bad plaster</td>
</tr>
<tr>
<td>24</td>
<td>Stone and plaster loss within the western upper edge of the roof façade</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Bad construction</td>
</tr>
<tr>
<td>25</td>
<td>The lower northern (right) strip of the wall is covered by marble mosaic adjacent to a sealed entrance blocked by red brick. The side is provided by two wooden doors of shallow wardrobes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>Mortar loss around the dislocated and failed mosaics</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Bad construction &amp; mortar that enhanced by the earthquake</td>
</tr>
<tr>
<td>27</td>
<td>Plaster loss from wooden lintel</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Bad construction &amp; mortar that enhanced by the earthquake</td>
</tr>
<tr>
<td>28</td>
<td>Irregular bifurcated (Y-shape) cracked plaster</td>
<td>75</td>
<td>1-2</td>
<td>4</td>
<td>Bad construction &amp; mortar that enhanced by the earthquake</td>
</tr>
<tr>
<td>29</td>
<td>Vertical, straight to slightly irregular cracked &amp; losted plaster upon red brick in the corner of the sealed room entrance</td>
<td>218</td>
<td>1-4</td>
<td>5</td>
<td>Bad construction &amp; mortar that enhanced by the earthquake</td>
</tr>
<tr>
<td>30</td>
<td>Space and separation between the vertical wooden frame of the middle door of belt in walled wardrobe</td>
<td>240</td>
<td>3-4</td>
<td>10</td>
<td>Earthquake</td>
</tr>
<tr>
<td>31</td>
<td>Vertical slightly irregular loss plaster upon red brick</td>
<td>240</td>
<td>3-12</td>
<td>3</td>
<td>Bad plaster quality &amp; enhanced by the earthquake</td>
</tr>
<tr>
<td>32</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>33</td>
<td>Oblique crack plaster inside the wardrobe</td>
<td>140</td>
<td>1-3</td>
<td>5</td>
<td>Earthquake</td>
</tr>
<tr>
<td>34</td>
<td>The lower 3 ms are covered by marble mosaics, the upper half is made of plaster limestone blocks</td>
<td>70</td>
<td>1</td>
<td>3</td>
<td>Earthquake</td>
</tr>
<tr>
<td>35</td>
<td>Mortar loss around the dislocated &amp; failed mosaics</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Bad construction of plaster &amp; Earthquake</td>
</tr>
</tbody>
</table>
The significant data obtained from Table 2.5 can be summarized as following:

1) Many architectural materials are seriously damaged in the five category’s sites particularly the plaster, less stone blocks and frontones in the ground floor façade, the stone blocks and lintels of the windows in the gate façade, the plasters and mosaics in the western and eastern side walls, the wooden balcony and capital, base’s decoration of the columns in the opened northern side of the Mekaad room.

2) The damage patterns are mainly observed as dangerous erosion corrosion, deformation, leaning and displacement of the stone blocks of the ground floor façade and inside the workshop. The entrance piers of the workshop show significant lateral deformation and leaning cracking in the causing, in turn, some adjacent walls.

3) The observed damages of the lower part of the building seem to be due to the subsidence in the foundation soil, moisture and rising of the subsoil water as it was explained. However, such damages were enhanced by human induced activities and earthquakes.

4) The plaster cracking and mortar loss that observed in the walls of Mekaad room are mainly due to the restoration deficiency and mortar used.

In conclusion, it is indicated that the cracks and loss in the plastered wall, displacement, separation, dislocation of the facade’s stone blocks in different category’s sites are partly influenced by previous earthquakes as well as by other causes (e.g. stone and plaster -mortar characteristics, and possible active deterioration process).

2.4.6 Information on local seismicity

2.4.6.1 Local seismicity

Earthquakes can result in damage of buildings to an unlimited extend based on the magnitude and duration of the quake itself as well as on the building characteristics that including architectural setting, building material, foundation soil condition and damage state category. However, the present section discusses the historical seismicity survey, the effects of the October 12, 1992 Cairo earthquake and finally the currently monitoring situation with emphasis on the damage evolution.

Although Cairo is not known to be in an active seismic zone, it is at least affected by seven large earthquakes in its recorded history (Maamoun, 1979); 1848 and 1992 earthquakes which appears to have originated at the same location (El-Baz personal communication), are the famous ones. According to the available documents and references (e.g. Maamoun, 1984; Deif, 1998; Said 1990; CEHM, 1999; Hussein, 1999; ENSN, 2009), the following maps of the historical earthquakes distribution were prepared:

Figure 2.109 shows the historical earthquakes from 2200 B.C. to 1900 A.D and in Shudan 1949 earthquake. Figure 2.110 shows the location of the instrumented earthquakes during the period
1900-1997 including the Aqaba earthquake at 1983. Figure 2.111, Figure 2.112 and Figure 2.113 respectively, show the distribution of the earthquake epicenters. Figure 2.114 shows the Location of the Cairo (October) earthquake at 1992.

Figure 2.109 - Intensity distribution of earthquakes of A, August 1847; B, 24 June 1870; C, 12 September 1955 and D, 31 March 1969 (A, B, and C after Maamoun 1979, and D after Maamoun & El-Khashab 1978). Earthquake intensity=IV - VII.

Figure 2.110 - Location of the instrumented earthquakes during the period 1990-1997 (after NRIAG, 2000).

Figure 2.111 - A. Location of permanent seismic stations and epicenters of historical and recent medium to large earthquakes. B. epicenters of small earthquakes.

Figure 2.112 - Epicentral distribution of all earthquakes, focal mechanisms of principal earthquakes and active seismic trends.

Figure 2.113 - Local seismic activity recorded by ENSN in Egypt from August, 1997 to 2005.

Figure 2.114 - Epicenter of the Cairo earthquake (solid circle). Open triangles represent the network station location.

According to the above mentioned seismicity maps the activity tends to occur along three main seismotectonic active trends; these are:
The northern Red Sea- Gulf of Suez - Cairo - Alexandria Clysmic trend.
The East Mediterranean - Cairo - Fayum Pelusiac trend and
The Levant - Aqaba trend. Accordingly, Cairo lies on the intersection of the two important structural trends (see figure 6-4) : the East Mediterranean - Cairo Fayum trend and the Red Sea - Suez Gulf - Cairo - Alexandria trend (Maamoun,1979).

2.4.6.2 Characterization of the seismic action

The Cairo Earthquake resulted severe damages in many archaeological buildings within old Cairo including the study Mekaad Radwan building. The magnitude of the deformation phenomena resulted from earthquake on the previous buildings damage situation before the quake type of building materials, Foundation soil condition and construction site height. The type of the foundation subsoil is to an extent, a controlling factor. It is clearly noted that the sites which built on alluvial clay (mud) foundation soil (like the case study) are more affected by the quake, than those built on limestone bedrock. The simulated Peak Ground Acceleration (P G A) is generally affected by the type of soil at the bedrock (Figure 2.113).

The foundation soil in site of the Mekaad Radwan is mainly sandy clay silt, and therefore the expected moderately to high ground motion can be noted. Furthermore, the damages could be also resulted from failure of the foundation water saturated soil due to the possible effect of soil liquefaction. Another influenced factor is the type of building materials and the applied technique structure is unreinforced masonry structure and was built by using the masonry limestone blocks and timber as the most common building materials that historical age, and therefore the building is mostly vulnerable during the Cairo earthquake. The Mekaad Radwan masonry was designed for vertical loads and since masonry has adequate compressive strength, the structure behaves well as long as the loads are vertical. However, when the masonry structure was subjected to lateral inertial loads during the earthquake, the walls develop shear stresses. The strength of the masonry under these conditions often depends on the bond between the limestone blocks and the used lime or gypsum motors. The bond of the mortar between the blocks is rare. The external plaster-less walls (the facades), the plaster internal side walls and mosaic covered were subjected to cracking and consequently to the mortar - plaster loss. The acceleration direction recorded during the October - Cairo earthquake suggests that the shock occurred mainly along the Eastern Mediterranean - Cairo Fayum trend. This means that the internal structures within building that are extended in this direction (NE-SW) would be more affected by the quake than others located in the right angle. Therefore the western and eastern side walls of the Mekaad radwan meeting room, which nearly extend in the NNE-SSW direction, have suffered more from the in-plane shear stresses, and consequently, were subjected to more oblique cracking and plaster loss in comparison to the plaster loss in comparison to the other walls in the study building. The acceleration directions recorded during the event suggest that the shock occurred mainly along the East Mediterranean Cairo- Fayum line. The October (Cairo) 1992 earthquake took place in Dahshor region, 25 km SW of Cairo city (M= 5.4-5.8). This earthquake is called the Cairo earthquake (Figure 6-6) by the National Research Institute of Astronomy and Geophysics.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

(NRIAG- Egypt). Its epicenter is located at a depth of 22 Km (NEIC) Cairo- Fayum line the East Mediterranean. This is the largest earthquake known to have occurred in this seismic zone since the historical destructive earthquake of August 1848. More than 8300 buildings were damaged or destroyed. For, this reason its damage effects on old Cairo including the case study monument are emphasized.

2.4.7 Previous and on-going studies

2.4.7.1 Research on materials

The building materials of the Mekaad Radwan monument are mainly composed of the masonry stone, mortars, plasters, and other decorative elements (such as marble columns and mosaic wall). The structure is made of timber, masonry load bearing walls and columns (Figure 2.116). The floor and ceiling are supported by timber beams, where as window and door opening are strengthened with timber or stone lintels. The façade of the building are composed of non-plastered walls made of large well-edge parallelepiped and/or cubed limestone blocks with regular horizontal and rare vertical mortar joints (Figure 2.117). The floor of the Mekaad room is covered by flattened nearly squared limestone tiles (Figure 2.118). The internal masonry load bearing walls are covered by lime-gypsum plasters (Figure 2.119). The lower parts of these walls are occasionally covered by decorated marble mosaics (Figure 2.120). The entrance of Mekaad room was sealed by red bricks and mortar and partially covered by plaster (Figure 2.119). Some secondary architectural elements represented by wooden balcony are shaded by decorated ceiling (Figure 2.121).

![Figure 2.116](image1.jpg) - Dark colored limestone blocks of the ground floor and side gate façade of the building.

![Figure 2.117](image2.jpg) - Limestone blocks and ground tiles of the western wall inside the Mekaad room. Note: the macrofossils e.g. pelecypod and gastropod.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

Figure 2.118 - Limestone flattened (ground) tiles of the Mekaad room.

Figure 2.119 - Red bricks sealing the entrance of the Mekaad Radwan room partially covered by plaster.

Figure 2.120 - Mosaic ornament covering the lower half of the eastern side wall of the Mekkad room.

Figure 2.121 - Decorated ceiling of the balcony cover showing some exfoliation and discoloration.

Table 2.6 - The identification minerals and their relative abundance in the limestone blocks of Mekaad Radwaan.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Symbol</th>
<th>Relative abundance of the minerals</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Calcite</td>
</tr>
<tr>
<td>Foraminiferal Lime-mudstone to</td>
<td>LSC</td>
<td>88%</td>
</tr>
<tr>
<td>Wakestone</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dolomitic Packstone</td>
<td>RLS</td>
<td>68%</td>
</tr>
</tbody>
</table>

Table 2.7 - The minerlogical result of x-ray diffraction analysis of the studied mortar and plaster samples.

<table>
<thead>
<tr>
<th>No type</th>
<th>Symbol</th>
<th>Relative abundance of the identified minerals %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar samples</td>
<td></td>
<td>Calcite</td>
</tr>
<tr>
<td>Limestone-sandy mortar</td>
<td>RBM</td>
<td>68 %</td>
</tr>
<tr>
<td>Lime-gypsum mortar</td>
<td>LSM</td>
<td>32 %</td>
</tr>
<tr>
<td>Gypsum mortar</td>
<td>ZLSM</td>
<td>12 %</td>
</tr>
<tr>
<td>Gypsum mortar</td>
<td>MME</td>
<td>11 %</td>
</tr>
<tr>
<td>Plaster samples</td>
<td></td>
<td>Calcite</td>
</tr>
<tr>
<td>Lime-gypsum plaster</td>
<td>RBP</td>
<td>51 %</td>
</tr>
<tr>
<td>Lime-gypsum plaster</td>
<td>LSP</td>
<td>45 %</td>
</tr>
</tbody>
</table>

Several samples were collected to present the limestone blocks and tiles as well as mortar and plaster. The petrographic investigation of the collected sample was carried out using thin sections.
and polarizing microscope. Bulk mineralogical identification was done through the interpretation of the XRD charts, the results are given in Table 2.6 and Table 2.7.

**Stone Masonry**

The masonry stones are suffering from cracking and micro porosity through which salinization and chemical weathering taking place. The present study investigated the rock petrographically and mineralogically properties and the decomposition situation of the limestones either due to natural digenetic fractures and microporosity or to the effects of surface karstification that represented by cavities and salinization. Petrographic investigation. The result of the microscopic investigation of limestone samples shown in Figure 2.122 to Figure 2.129, respectively, and their XRD analysis are shown in Figure 2.132 and Figure 2.133.

![Photomicrograph of the forminiferal lime-mudstone to wackestone with algal fragments and microtests embedded in lime matrix.](image)

![Photomicrograph of the forminiferal lime-mudstone to wackestone with few quartz grains, gypsum fragments and dolomite rhombs.](image)

![Photomicrograph of the dolomitic skeletal packstone with zoned idiotopic Fe-dolomite rhombs and microvoids in micrite matrix.](image)

![Photomicrograph of the dolomitic skeletal packstone with alveolinoid test and carbonate fragments embedded in microsparite with dolomite rhombs.](image)

![Photomicrograph of the dolomitic skeletal packstone with alga fragments, micrite pellets and scattered dolomite rhombs in sparry micrite matrix.](image)

![Photomicrograph of the dolomitic skeletal packstone with ankerite rhombs that have dark cone iron oxide and clear outer rims. Notice the digenetic pore space.](image)
Petrographically, three types of limestones were identified within the architectural elements. The first rock type (Foraminiferal lime-mudstone to wacked -stone ) Figure 2.122 and Figure 2.123. The most common type consisting the limestone blocks of the internal western and eastern walls of the Mekaad Radwaan meetings room Figure 2.117 and Figure 2.120 and is represented by sample (LSC) . The second rock type (Dolomitic skeletal packstone) Figure 2.124 to Figure 2.127 is very common in facades of the ground floor and the site gate of the building Figure 2.116 and is represented by sample (RLS). The third rock type (Skeletal Packstone to Grainstone) Figure 2.126 to Figure 2.129. It is very common type in the ground tile of the Mekaad Radwaan meetings room Figure 2.118.

**Brief petrographic and distribution of the identified limestone rock types**

1) Foraminiferal Lime - Mudstone to Wakestone

This rock type represents the chalky yellowish white with dark patches limestone blocks in the masonry walls. It ranges from foraminiferal lime -mudstone to wake stone showing micro porosity
with numerous dissolute pores and microcavities. The skeletal particles are mainly represented by benthonic and minor planktonic foraminiferal tests in addition to some algal fragments. These particles are randomly scattered within miccrystalline lime mud matrix (Figure 2.122 and Figure 2.123). Very few very fine quartz grains, traces of gypsum particles and dolomite rhombs could be observed in some parts of the rock. The results of the XRD analysis (Figure 2.132) support the mineral composition of the rock that revealed from the thin section investigation (Table 2.6). Table 2 indicates that the most abundant carbonate mineral is calcite (88%) with traces of gypsum (4%), quartz (6%) and Dolomite (2%).

2) Dolomitic Skeletal Packstone
The rock type is reddish in color with clear dark patches composed of foraminifera tests, algal fragments and pelecypoda shell fragments imbedded in dolomitic micrite which partially recrystallized into microsparry calcite (Figure 2.124 to Figure 2.127). The rock has numerous pores, shrinkage microcracks and microvoids (Figure 2.124). Some voids are shown with partially filling of zoned idiotopic ferron dolomite or ankerite rhombs (Figure 2.124 and Figure 2.125). Some other rhombs are scattered in a reddish lime mud matrix. The ankerite rhombs have dark core of iron oxide and clear outer rims (Figure 2.124 and Figure 2.127). The chambers of the foraminiferal tests are filled with sparry calcite and their walls are heavily micritized. The pelecypod shell fragments are originally preserved in its fibrous texture (Figure 2.125), although some parts of them are partially recrystallized to sparry calcite. Some concave-convex grain mechanical compaction (Figure 2.125). In this rock dolomitization and neamorphism are the most digenetic features. The pore spaces observed in the reddish dolomitic parts of the rock (Figure 2.127) may be due to the dissolution of the carbonate crystals as later stage of diagnosis. The mineralogical composition as interpreted from the XRD analysis (Figure 2.133-Table 2.6) proved that the rock is essentially composed of calcite (68%) followed by ankerite (21%) with traces of quartz (7%), gypsum (2%) and halite (2%).

3) Skeletal Packstone to Grainstone
This highly fossiferous limestone is very common in tiles used at the ground of Mekaad Radwan room. It is mainly made up of skeletal particles, bioclasts (e.g. shell fragments of pelecypod and occasionally gastropod as well as large foraminiferal tests) cemented by micr sparite or embedded in lime mud matrix (Figure 2.128 to Figure 2.131). Some of the bioclasts are rimmed by micrite envelopes and their cavities are filled with errant sparry calcite while others are partially or completely micritized. The chambers of some tests are filled with sparry calcite while wall still preserved without aggreading neamorphism. Isopachous calcite crystals exist on the boundary of some bivalve shell fragments (Figure 2.130).

Compression strength testing
The geotechnical properties as given in (Table 2.8 and Table 2.9) for the limestone block and red brick (dry and wet) respectively indicate that the limestone samples are more resistant than the red brick samples. The limestone is denser and less porosity than the brick samples.

Table 2.8 - The physical properties of the building limestone of Mekaad Radwan (dry status).

<table>
<thead>
<tr>
<th>Samples</th>
<th>Length (cm)</th>
<th>Width (cm)</th>
<th>Height (cm)</th>
<th>Weight (dry)</th>
<th>Porosity (%)</th>
<th>Density (%)</th>
<th>Compressive strength (Kg/cm²)</th>
<th>Load (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.5</td>
<td>5.1</td>
<td>4.8</td>
<td>231.2</td>
<td>16</td>
<td>2.10</td>
<td>92.9</td>
<td>20.9</td>
</tr>
<tr>
<td>2</td>
<td>5.0</td>
<td>4.5</td>
<td>4.7</td>
<td>216.3</td>
<td>16</td>
<td>2.05</td>
<td>95.7</td>
<td>21.1</td>
</tr>
<tr>
<td>3</td>
<td>5.0</td>
<td>4.5</td>
<td>4.9</td>
<td>217.6</td>
<td>16</td>
<td>1.97</td>
<td>100.2</td>
<td>22.1</td>
</tr>
<tr>
<td>Average</td>
<td>4.8</td>
<td>4.7</td>
<td>4.8</td>
<td>221.7</td>
<td>16</td>
<td>2.0</td>
<td>96.2</td>
<td>21.4</td>
</tr>
</tbody>
</table>

Table 2.9 - The physical properties of the building limestone of Mekaad Radwan (wet status).

<table>
<thead>
<tr>
<th>Samples</th>
<th>Length (cm)</th>
<th>Width (cm)</th>
<th>Height (cm)</th>
<th>Weight (dry)</th>
<th>Weight (dry)</th>
<th>Absorpt -ion %</th>
<th>Compressive strength (Kg/cm²)</th>
<th>Load (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.5</td>
<td>5.1</td>
<td>4.8</td>
<td>231.2</td>
<td>16.5</td>
<td>2.05</td>
<td>92.9</td>
<td>20.9</td>
</tr>
<tr>
<td>2</td>
<td>5.0</td>
<td>4.5</td>
<td>4.7</td>
<td>216.3</td>
<td>16.6</td>
<td>2.03</td>
<td>95.7</td>
<td>21.1</td>
</tr>
<tr>
<td>3</td>
<td>5.0</td>
<td>4.5</td>
<td>4.9</td>
<td>217.6</td>
<td>16.8</td>
<td>1.98</td>
<td>100.2</td>
<td>22.1</td>
</tr>
<tr>
<td>Average</td>
<td>4.8</td>
<td>4.7</td>
<td>4.8</td>
<td>221.7</td>
<td>16.7</td>
<td>2.01</td>
<td>96.2</td>
<td>21.4</td>
</tr>
</tbody>
</table>
Conclusions

The field observation and petrographical and mineralogical investigation of the studied limestone samples indicate that nearly all the masonry limestone blocks and tiles of the study monument come from the famous quarry regions of the Mokattam Plateau, bordering old Cairo from the southeast. Both the limestone building elements and the quarry layers belong to the Mokatam Formation of the Middle Eocene age. This formation is mainly composed of hard chalky limestone with microfossils (e.g. Nummlite or Alveolina tests). The limestone blocks may differ a little in their mineral composition and texture; the parameters which determine their technical properties and weathering behavior. Consequently, the study masonry stones suffer from cracking and microporosity, allowing salinization and chemical weathering to take effect in different degrees and patterns.

2.4.7.2 Mortar and Plasters

Mortar and plaster are of the most effected building and decorated materials by damage and deterioration factors (Figure 2.134 and Figure 2.135). The analysis of mortar and plaster samples is an important step before any restoration intervention. So, the samples must be taken and prepared carefully for physical, mineralogical and petrographic analysis. In the present case study several samples were collected from ancient and resent mortars as well as from recent plaster materials that were used during the last restoration upon either limestone blocks or the red bricks. However, some physical characteristics are impossible to evaluate due to the difficulty to do rigorous determinations with irregular, friable specimens cutting standard sized test sample that could be collected from very soft ancient mortar. Table 2.10 illustrates the specific locations of the mortar and plaster samples within the different elements of the study monument Mekaad Raddwan as shown in Figure 2.134 and Figure 2.135.

Table 2.10 - The studied specific locations.

<table>
<thead>
<tr>
<th>Type</th>
<th>Symbol</th>
<th>Specific Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar Samples</td>
<td>MBM</td>
<td>Binding mortar between red bricks</td>
</tr>
<tr>
<td></td>
<td>LSM</td>
<td>Binding mortar between limestone blocks</td>
</tr>
<tr>
<td></td>
<td>ZLSP</td>
<td>Binding mortar between zigzag limestone blocks</td>
</tr>
<tr>
<td></td>
<td>MME</td>
<td>Collating mortar of the mosaics</td>
</tr>
<tr>
<td>Plaster samples</td>
<td>MBP</td>
<td>Plaster upon the red brick</td>
</tr>
<tr>
<td></td>
<td>LSP</td>
<td>Plaster upon the limestone blocks</td>
</tr>
</tbody>
</table>

Petrographic investigation

The microscopic investigation prove that the plaster sample (LSP) located upon the limestone blocks is similar to the mortar sample that binding the same blocks (Figure 2.136 and Figure 2.138).

Both are composed of lime lumps represented by fine calcite particles with gypsum, few anhydrite crystals, and few fine quartz sand grains (Figure 2.136). Some additives (very fine fibrous and organic materials) are also observed embedded in the gypsiferous lime matrix. Some gypsum
particles seem too altered to anhydrite crystals (Figure 2.137). The XRD results support the composition of the plaster sample (LSP).
The XRD result of mortars (Figure 2.138 to Figure 2.143 and Table 2.11) indicated that there are three types:

- The first is lime mortar type representing by samples RBM which used as binding material between the red brick. The first type mainly consists of calcite (68%) and gypsum (7%) with quartz (25%) (Figure 2.138).

- The second type is lime-gypsum mortar respective stone blocks sample LSM. The second type was used as binding material for the building limestone blocks contains gypsum (24%), calcite (32%) with excess quartz (35%) in addition to some feldspar grains (9%). Such feldspar (mainly k-feldspar) associated with a part of quartz may be derived from granitic rock powder which possibly was added as additives to ancient mortar (Figure 2.139).

- The third type gypsum mortar used as binder representing by sample ZLSM and MME which was used as collating or binding of the zigzag decoration elements and mosaics. The third type consists of gypsum (72-76%), little calcite (11-12%) and traces of quartz (5-6%).

The weathered gypsum mortar usually includes 8-10% salt (sodium chloride) which is called in the X-ray chart "synthetic halite". This salt is not considered as one of the components of the mortar, but it was formed from salinization weathering of mortar as it will be explained in section (5). That salt has a negative influence on the quality of the mortar. Therefore it is unlikely an additive. The plaster samples are lime-gypsum type. The X-ray analysis of the plaster samples (RBP) and (LSP) shows that they are composed of (45-51%) calcite (lime), (39-44%) gypsum and traces of quartz (5-8%) as given in Table 3.4. Some ancient mortar or plaster, such as sample (LSP) occasionally contains few anhydrite crystals (8%) that may be formed by dehydration of the original gypsum at higher temperature (see Table 2.11).

<table>
<thead>
<tr>
<th>Mortar samples</th>
<th>No type</th>
<th>Symbol</th>
<th>Relative abundance of the identified minerals %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Calcite</td>
<td>Gypsum</td>
</tr>
<tr>
<td>Limestone -sandy mortar</td>
<td>RBM</td>
<td>68 %</td>
<td>7 %</td>
</tr>
<tr>
<td>Lime-gypsum mortar</td>
<td>LSM</td>
<td>32 %</td>
<td>24 %</td>
</tr>
<tr>
<td>Gypsum mortar</td>
<td>ZLSM</td>
<td>12 %</td>
<td>72 %</td>
</tr>
<tr>
<td>Gypsum mortar</td>
<td>MME</td>
<td>11 %</td>
<td>76 %</td>
</tr>
</tbody>
</table>

Conclusions
It is concluded that the determination of the different components proportion and the study of their deterioration and damage situation is very important before any restoration of mortar and plaster. Some samples have few content of sodium chloride which has negative influence on the quality of the mortar. Most of the samples investigated from the case study mortars and plaster contain both
gypsum as well as calcite in the form of limestone powder. Climatic action together with high moisture levels are usually the important causative factors for the deterioration and mortar-plaster loss. However the interior composition particularly the proportion of gypsum and sand additives of the mortars is another significant factor controlling the level of cracking and losses. Generally in preparation of mortar or plaster for restoration processes, the addition of gypsum improves the strength of lime - various pozzolan mixes, but the gypsum content should remain limited because the ettringite formation causes swelling and disintegration of the material (Toumbakari 2002). Sometimes, it is recommended to use lime/sand mortar, like that of sample (RBM) because it is more appropriate for repair of older masonry with an existing lime mortar, and where exposure to climate loads is low. It is important to avoid the presence of soluble salts, or salt crystals in the prepared mortar for restoration and therefore avoiding the appearance of efflorescence and subflorescence. These properties can be relatively modified by altering the process of production of the mortar, the type of aggregate, the aggregate/binder and water/binder ratio…..etc. (Palermo et al., 2004).
2.4.7.3 Other inspection works

The main damage causes of Mekaad Radwaan monument and the surrounding archaeological site as well, are the properties of the building material, characteristics of the foundation soil, and social features of the environment in addition to the occasional ruinous events of earthquakes. The present section discusses the condition necessary for salt weathering processes through investigation of the possible sources of subsoil water and air humidity.

Characteristics of foundation soil (Texture and Minaralogy)

The ground floor of Mekaad Radwann building consists of a large storage room taking the whole building area. The building is based on the thick basement load bearing walls and pillars constituting the foundation structure. The foundation appears to be very shallow and the foundation soil is exposed in some parts of the floor (Figure 2.144 and Figure 2.145). In these figures the soil appears to be welled by sub-soil water. The ascending moisture from the ground water causes dangerous deterioration problems not only for the whole foundation structure but also is up to six meters above the ground surface. The soil foundation of some Islamic monuments at old Cairo has been the subject of several investigation carried out before and after the October 1992 earthquake. As shown in some previously studied bore holes with depth ranging between 7 and 20 meters, the subsoil are mainly formed of surface fill layer (3-4 meter thick) composed of the Nile flood silts and clay matrix mixed with fine to medium sand with traces of limestone fragments. The alluvial soil which is very heterogeneous in thickness (ranging from 5-20 meter) is commonly underling by Eocene limestone bed rock. The foundation soil underneath the study building was mechanically (Figure 2.145) and clay mineralogically analyzed (Figure 2.147). The results of the mechanical analysis indicate that the soil texture is mainly fine sandy clayey site with 15% fine and very fine sand, 71% silt and about 14% clay (Figure 2.146). In this respect it very similar to the matrix of the reworked heterogeneous fill layer that covering the Nile alluvial deposits.
As mentioned above the subsoil water level is very high and can be observed within, surface soil of foundation inside the workshop room in the ground floor of the Mekaad Radwan building Figure 2.148. Old Cairo is suffering from the rising of the water-table that has affected the whole urban and historical areas during the last decade due to different reasons of which:

1) The hectic growth of population and leakage of the domestic water and sewage system.
2) The possible infiltration of the excess drainage water coming from the nearly green areas (e.g. gardens).
3) The possible lateral and upward seepage of water from the shallow angrier and River Nile.
4) The impervious concrete barriers and tunnels of underground metro that currently carrying out may prevent the natural flowing of groundwater and subsequently tedding to excessive rise in subsoil water.
5) The possible leakage of water that may be coming from the limited (local) industrialization which is adding environmental decay cause.

The damage and degradation process that affecting nearly all Egyptian monuments is caused mainly by the crystallization of various salts. A significant parts of building stones, bricks, mortar and plaster of the Mekaad Radwan building were suffered from salt-weathering and salinization process. This disintegration process is caused by the crystallization of various salts, mainly sodium chloride underneath the surface layers of the masonry element (e.g. stone). Occasionally under certain physical conditions these salt, crystals are brought
to the surface of the stone as shown in Figure 2.148. For such crystallization to take place three conditions are necessary:

1) The presence of water - soluble salts (e.g. sodium chloride).
2) The presence of water to dissolve the salts.
3) The opportunity for the salts to be brought to the surface of the stone and crystalline out by the evaporation of the water holding them in solution.

To confirm the identification of the type of the result, X-ray analysis was carried out on a salt sample shown in Figure 2.149. XRD result indicate the dominance of sodium chloride crystals (halite ,synthetic ) with about 74% with some contamination of accessory material like quartz as sand 12% ,calcite (as lime powder ) and synthetic gypsum . Such contaminants may be derived from the mortar or plaster used in this wall.

Occasionally, some fine limestone and gypsum fragments are observed within the foundation sub soil. The foundation soil is highly wetted by slightly saline water Figure 2.144. The clay minerals identification depends upon the XRD analysis of the clay fraction which constitutes of representative soil sample and its repose to glycolation and heat treatment as mentioned in section one. The XRD result is illustrated in Figure 2.149, XRD data reveal that the foundation soil clay contain predominantly smectite with small proportion of kaolinite that associated with few quartz ,calcite and gypsum .This result may be coincide with clay mineral content of Nile Delta that was previously examined . According to the presence of little clay component (about 14% of this sample), the foundation soil is generally considered as slightly expandable sandy silt deposits on wetting from the subsoil water. Based on the previous studies concerning the salt weathering process, the data analysis has essentially shown two kinds of action by the moisture:-

1) The absorption of humidity from the air by the salts inside the walls and.
2) The rising moisture from the ground water or subsoil water.

In the present study the sources of salts inside the walls are mainly coming from natural or synthetic salts included in the limestone blocks, the red brick as well as in mortar or plaster as explained in the above mentioned sections. The source of rising moisture is coming from the subsoil water at the foundation level. The availability of this condition may increase the quantity and dampness height within the ground floor of the study monument. Generally the quantity and height of dampness resulted from the subsoil water depend on the depth of the groundwater -table ,the lower level and the upper part of the buildings ,the porosity of the materials and the width of suction cross-section upon which the capillary suction of dampness from the ground is depended .

- Geoenvironmental impact
The Mekaad Radwan site is suffering from not only salt weathering but also from some disturbance due to external works and human activities. The external work (e.g. misuses of historical building nearby constructions) and human activity (e.g. workshops and other commercial activities) may be cause slowly by time significant alterations to the pattern of stresses in the structural element of building.

The following is brief account on the impacts of moisture, humidity, salt weathering and foundation condition as well as the human induced activities as illustrated from some examples of dramatically deteriorated architecture elements (Figure 2.150 to Figure 2.162, respectively).

1) External pulverization, swelling, block siding falling and collapsing of walls ceiling, ground of the workshop site at the ground floor of the building (Figure 2.150 to Figure 2.153, respectively)

2) Appearance of cracking missing of plaster layer, Salinized spots and disintegration efflorescence. Salt crystal aggregates scattering on red brick, mortar, mosaic, ground tile, column’s base (Figure 2.154 Figure 2.161, respectively).

3) Deformations imposed on the foundation as a result of soil subsidence which may cause lesions or aggravation of an existing pattern of cracks, according to the stresses induced in the bearing elements (e.g. the deformation of the workshop site, shown in Figure 2.150 to Figure 2.153, respectively).

4) Deterioration (karstification, exfoliation erosion, corrosion) at the basis of the walls due to the rising of dampness by capillary suction from the subsoil water (Figure 2.160 and Figure 2.161).

5) Missing of decoration features (Figure 2.162), discoloration, pitting, scaling and exfoliation the wooden ceiling of the balcony.

Figure 2.150 - External pulverization, block sliding and collapsing.

Figure 2.151 - Salinization and swelling of workshop's walls.

Figure 2.152 - Deformation and salinization of the workshop's ceiling.

Figure 2.153 - Scattering of the salt crystals on the restorated stones inside the main entrance of the building.
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Figure 2.154 - Salt aggregates in the cavities of stones inside the building's gate.

Figure 2.155 - Deterioration and mortar loss on the red bricks inside the main entrance.

Figure 2.156 - Cracking of the mortar crust on the entrance's wall.

Figure 2.157 - Salinization and corrosion of the mortar between the red bricks inside the wardrobe.

Figure 2.158 - Scattering of the salt crystals and mortar loss on and around the mosaics of the eastern wall.

Figure 2.159 - Karstification and corrosion of the tiles mortar inside the Mekaad room.

Figure 2.160 - Salinization and missing of the decoration of the column base.

Figure 2.161 - Missing of the decoration of the marble column.

Figure 2.162 - Scaling and exfoliation of the wooden ceiling of the balcony.
2.4.8 Proposed inspection works

The current primitive monitoring method, which depends on direct observation using a gypsum spy, has detected some severe evolutionary separation and dislocation of the masonry blocks in the gate facade and arch. It is highly recommended to monitor the evolitional cracks and other damage phenomena in the study monument, which are mostly connected to a series of causative factors, such as:

- The rise of groundwater within the foundation soil
- The variation in temperature, causing thermal deformation
- The variation in humidity and moisture content
- The differential settlement of the foundation soil
- The weakening of the mechanical characteristics of the building material, especially the gypsum mortar and plaster
- The ground movements and possible liquefaction that could result from possible earthquakes

The monitoring of the evolutionary alteration in the architectural elements of the Mekaad Radwan case study is necessary to define the design of the strengthening work and proper restoration intervention. It is recommended to use lime-very fine sand mortar, because it is more appropriate with the existing lime mortar that has low climate load exposure.

Table 2.12 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Tasks already carried out or in course, with available results</th>
<th>Tasks to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1- Inspection</strong> (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Petrographical and mineralogical investigation of the building blocks, mortar and plaster samples, using polarizing microscopy and XRD techniques</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td>- The rise of groundwater within the foundation soil</td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>- The variation in temperature, causing thermal deformation</td>
</tr>
<tr>
<td></td>
<td>- The variation in humidity and moisture content</td>
</tr>
<tr>
<td></td>
<td>- The differential settlement of the foundation soil</td>
</tr>
<tr>
<td></td>
<td>- The weakening of the mechanical characteristics of the building material, especially the gypsum mortar and plaster</td>
</tr>
<tr>
<td></td>
<td>- The ground movements and possible liquefaction that could result from possible earthquakes</td>
</tr>
</tbody>
</table>

2.4.9 References

• Kamh GME (1994) The impact of geological conditions on the Islamic archaeological sites at El-Gammalia area, Cairo City, Egypt. MSc thesis, Faculty of Science, Menoufiya University, Egypt.
2.5 CHURCH OF STS PETER AND PAUL, PRAGUE, CZECH REPUBLIC

2.5.1 Name, location and description

The Vratislav II’s Kostel Sv Petra a Pavla (Church of Sts Peter and Paul) is located in Vyšehrad, in Prague, Czech Republic (Figure 2.163).

Figure 2.163 - Main facade.

2.5.2 Historical note

The history of the Church of St. Peter and Paul at Vyšehrad starts after the year 1070, when king Vratislav I founded the chapter house. According to Nechvátal (2000), the church has been rebuilt and modified since 1070, so its history could be divided in 7 periods (the time line is in the Figure 2.164).

I. Romanesque (First Stage)   V. Renaissance
II. Romanesque (Second Stage) VI. Baroque
III. Early gothic   VII. Neo Gothic
IV. High gothic

The seventh building stage was a neo-Gothic reconstruction, which has been preserved up to the present. It took place between 1885 and 1903 according to plans of the master builder, Joseph Mocker. Regarding the bells, they have been preserved from the original Renaissance section of the building, and they now grace the present neo-Gothic church.

Figure 2.164 - Time Line.

2.5.3 Historical and artistic significance

The Vratislav II’s Kostel Sv Petra a Pavla (Church of Sts Peter and Paul) is the architectural historic monument in the Czech Republic. It is also one the largest and structurally more outstanding medieval buildings in Europe. In addition to its structural and architectural relevance,
Church of Sts Peter and Paul include a significant amount of valuable and immovable artworks produced from the Middle Age to modern times.

2.5.4 Structural and material features

The cathedral was built with traditional materials (stone, bricks, covered with slate on wooden truss). The building is made up of three naves (Figure 2.165, plan view); its total width is 30 m and the length 56. The height of the roof crest is 28 m. As it could be observed in Figure 1, there are two towers in the façade: both have the same height: 60 m. The square cross-section of each tower is 7 x 7 m, the bells are hung in classical wooden stools placed at the level about 29 m. There are two old bells in the southern tower (J1, J2); four recently mounted ones in the northern tower (S1 - S4). All bells exert swaying motion driven separately by electric motors. Besides, there is a set of 12 stable bells with clappers controlled by computer in northern tower, used for playing some song. In the Church Sts Peter and Paul, the triangular gable wall over the main entrance starts at the height of 19.60 m above the church floor, its width between both church towers is 9.0 m and the height of its rectangular part fixed into the towers is 2.60 m. The height of the freestanding triangular part is 8.70 m and the cross on its top reaches the height 30 m. The gable wall does not lean against the wooden construction of the roof covering the main nave of the church. The thickness of the masonry of the gable is 0.57 m, but the niches containing statues reduce it to 0.27 m.

The material of the walls is masonry mainly constituted by stone and lime mortar. The thickness of the masonry of the building varies between 0.3 and 1.2 meters; it depends where the masonry is located in the building. Basically the thickness of the lateral walls is 100 cm, and the thickness of the walls that are in the façade is 120 cm. Regarding the mechanical properties, conservative values for this masonry are: specific weight 22 KN/m$^3$ and elastic modulus 4 GPa. Between the two towers, it is also possible to see the gable, but its description is in the next section. The church interior was recently refurbished and also its structure seems to be in good order and condition (Figure 2.166). In the lateral view (Figure 2.167), it could be seen that also the state of the walls are in good condition.
2.5.5 Present and foreseen future use. People at risk.

The church has been used, since its construction, for the corresponding liturgical purposes. However, and since the popularization of Prague as a tourist destination, the Church has become nowadays a powerful cultural attraction, hosting several thousands visitors each year. Many people visit the building each day. In addition, liturgy and cultural activities are regularly celebrated.

2.5.6 Considerations on valuable cultural contents

The church includes significant artistic contents including both movable and immovable works of art, such as sculptures, paintings and objects. The most remarkable works of art are:

- Historical movable heritage such as a medieval artistic liturgical objects
- Medieval stone sculptures and decoration in the gable.

Figure 2.168 - Main facade and detail of the gable.

Figure 2.169 - Roof detail.

Figure 2.170 - Interior.
2.5.7 Present condition and damage

2.5.7.1 Considerations on the present condition
In spite of some existing damage, the structure is well preserved and no severe problem or active deterioration process has been identified. The structure has been subjected to continuous maintenance and repair in medieval times.

2.5.7.2 Main observed damage and decay
It was observed that the triangular gable wall in the façade, due to the ringing of the bells, started to vibrate out of its plane. In the year 2002, a study of the church (ITAM, 2002) concluded that the swaying bells excite the structure with considerable horizontal displacements and with frequencies of one, two and three multiple of the bells motion. This excitation can be amplified by resonance effect of an arbitrary structural element, which is tuned to one of these frequencies (see Figure 2.171).

![Figure 2.171 - Bell.](image)

2.5.8 Information on local seismicity
The Czech Republic is seismic zone characterized by peak ground accelerations <0.1g (maximum 0.085g).

2.5.9 Previous and on-going studies

2.5.9.1 Introduction
Previous detailed studies involving inspection, monitoring and structural analyses have been carried out by ITAM (Institute of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic) in the year 2002 and 2007.

2.5.9.2 Geometrical survey
A detailed survey of the entire building is available. Additionally, detailed survey of deformation has been carried out.

2.5.9.3 Research on materials
The material of the walls is masonry mainly constituted by stone and lime mortar. Regarding the mechanical properties, conservative values for this masonry are: specific weight 22 KN/m³ and elastic modulus 4 GPa.

2.5.9.4 Visual inspection
Detailed visual inspection has been carried out on the entire building for the purpose of identifying morphology, materials and damage.

2.5.9.5 Monitoring
The data that was analysed corresponds to the measurements on the church building that were done by ITAM (Institute of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic) in the year 2002 and 2007. It was observed that the triangular gable wall in the façade, due to the ringing of the bells, started to vibrate out of its plane. Because of this situation, the dynamic properties of the church were measured with accelerometers in 2002, and then it was concluded that the gable was in resonance with the ringing of the bells, so it was necessary to retrofit it. In the year 2007, after the retrofit of the gable, again the dynamic properties of the church were measured, and it was concluded that the gable was not in resonance with the ringing of the bells. The values of the dynamic characteristics of the building, before and after the retrofit of the gable, were obtained only using experimental techniques. The other possibility to obtain these values is with structural models. In this investigation these parameters were obtained with structural models, and then they were compared with the experimental values. If the mechanical properties and geometrical characteristics that are necessary to take into account for the model are conservative, and the boundary conditions are according to the real conditions, the values are going to be acceptable (in the tolerance range). In this specific case, the experimental results of the frequency of the historical building were very useful because they helped to calibrate the analytic model.

2.5.9.6 Structural analysis

In the structural model, because it was focused in study the response of the gable, before and after the retrofit, only a section of the structure was modelled, with Finite Element Analysis, using the software SAP 2000. For this structural model, the elements were shells (2D elements), the soil was considered as completely rigid, the masonry was assumed as homogeneous and isotropic, and the analysis was carried out only in the linear range. The calculated dynamic characteristics correspond to the eigen-frequencies values obtained with the dynamic analysis of the structural model. For the measured dynamic characteristics, the previous data (ITAM Reports) for the year 2002 (before the retrofit) and the year 2007 (after the retrofit) were taken into account. In both cases the ringing of the bells excited the structure, and then the frequencies were obtained analysing the results in the frequency domain. The results that were analyzed in this investigation were the values that were obtained due to the dynamic excitation caused by the bells J1 and J2.

Initially, it was used a simple model for the Gable, the problem with this model, was that it was possible to obtain a good approximation of the first mode, but for the other modes, second or third, the results were so far of the experimental value. Because of this situation, a model of the façade was carried out, and the elements where shells (2-D elements with thickness). The masonry corresponds to a historical construction, so to be concerned about the cracks and general damaged in the masonry, a value of effective inertia was taken into account. Because of this consideration, instead of a value gross value of the second moment of inertia (Igross) a value of effective inertia was calculated and it was part of the input in the mechanical properties. The relation of the effective Inertia (Ieff) and the gross Inertia (Igross), is established with the following equation:

\[ I_{eff} = \alpha I_{gross} \]

In this equation \( \alpha \) corresponds to a rigidity coefficient. This value of \( \alpha \) varies between 0.5 and 1. According to the results that were obtained, the values of frequency that were closer or at least in the tolerance range were when a value considered equal to 0.6, so the results that are presented correspond to this value of \( \alpha \).

For the model of year 2002, initially a model focus on the gable was performed. The problem with that model was the fact that the Eigen- frequency was 1 Hz, and then the values of the Eigen-frequencies for the modes two and tree were 4.5 Hz and 5.8 Hz. Values that if they are compared with the experimental values are so far away, specially for the second and third mode. The problem with this first model approach is related with the fact that the boundary conditions that were taken into account did not reflect the real situation of the connection of the gable and the towers. The restriction out of the plane was completely fixed, but the real situation is that there is stiffness associated due to the walls out of the plane and the mechanical properties of the masonry that influences it.
In this model was assumed that the restrictions out of the plane were fixed, but the real situation as it was commented, is that they could move out of the plane but with the restriction of the towers, walls out of the plane and the façade. Because the model that was performed did not bring good approximations, and the boundary conditions that were assumed did not reflect the real situation, it was decided to model a section of the building. In the Figure 2.172 is presented the section of the church that was considered. The reason of why also the towers were included in the model was that they could help to model better the restrictions out of the plane and also that in them are the bells and the spires, and they are considerable weighs that influence the result of the Eigen-values.

According to the recommendations of Lourenço, 2001, in this 3 D model, the elements that composed it are 2D shell elements. Once the geometrical configuration of structural model of 2002 was figured out, it was defined the thickness of each element. In the façade is possible to see that the thickness of the gable is 30 cm, and in its top is 60 cm, in order to considerer the fact that in this part has a spire. The sculptures that are in this area were not included because they are not linked to the gable. For the tower there is a section was the masonry is thicker, it is between the height of 19 and 29 meters. Because of that reason in the model is possible to see that in this section of the tower the colour of the elements is red (60 cm).

Another important fact that had to be considered was the openings, specially the arches over windows and door openings. Initially the arches were not considered, but once the first results were obtained, it was observed that they influenced the stiffness of the façade out of the plane. In the model those walls were review in order to represent as much as possible its influence in the restriction out of the plane of the façade. In the model the difference of the length of wall was considered; the thickness of those walls is constant.
2.5.10 Previous or on-going restoration works

2.5.10.1 General restoration works

In order to change the natural frequency of the gable, some steel plated were anchored in the gable and fixed to the structure of the roof. In the Figure 2.175(a) is presented in red, the position in the lateral view of these steel plates. In the Figure 2.175(b) is presented the detail of how the steel plate was anchored in the gable and then fixed in different timber elements. The objective was to take advantage of the stiffness of the timber truss out of the plane (this structural element is presented in the sketch in the Figure 2.175(c)).

With this work, it was possible to change the restrictions of the movement of the gable out of its plane, and as it was reviewed in the experimental tests it helped to avoid the resonance phenomena. Also in the structural mode, it is going to be possible to take into account this retrofit.
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Figure 2.175 - Details of the retrofit of the gable.

Table 2.13 - Description of the Position of the Points (Itam, 2002).

<table>
<thead>
<tr>
<th>Point No</th>
<th>Position</th>
<th>Structural Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>Southern tower top</td>
<td>Southern Tower</td>
</tr>
<tr>
<td>20</td>
<td>Southern tower above bells</td>
<td>Southern Tower</td>
</tr>
<tr>
<td>2</td>
<td>Southern tower bells level</td>
<td>Southern Tower</td>
</tr>
<tr>
<td>21</td>
<td>Southern tower gable base level</td>
<td>Southern Tower</td>
</tr>
<tr>
<td>22</td>
<td>Southern tower bottom</td>
<td>Southern Tower</td>
</tr>
<tr>
<td>6</td>
<td>Gable top</td>
<td>Gable</td>
</tr>
<tr>
<td>7</td>
<td>Gable low, centre</td>
<td>Gable</td>
</tr>
<tr>
<td>8</td>
<td>Gable low, right</td>
<td>Gable</td>
</tr>
<tr>
<td>9</td>
<td>Gable low, left</td>
<td>Gable</td>
</tr>
<tr>
<td>4</td>
<td>Gable base</td>
<td>Gable</td>
</tr>
</tbody>
</table>

(a) Lateral view
(b) Detail of the retrofit
(c) Detail of the timber truss

Figure 2.176 - Measuring Point in the Façade, Point 8.
Figure 2.177 - Measuring Point in the Façade, Point 10.
For each point, once its data was analysed, the Root Mean Square (RMS) of the displacements in millimetres (mm) and the frequency peaks in Hertz (Hz) were evaluated. As the registrations were more or less of the sinusoidal shape, the informative maximum amplitudes were checked as $\sqrt{2} \times$ RMS value.

So for instance, due to the dynamic excitation of the ringing of the bell J1, in 2002, it was possible to recover in the point 6 the graph that is presented in the Figure 2.178. In those graphs is possible to determine according to the spectral density (mm$^2$/s) the peak that correspond to the frequency related with the modes of structure.

![Graph](image)

Figure 2.178 - Frequencies and spectral densities at point 6 (year 2002) (ITAM, 2002).

### 2.5.10.2 Monitoring phases

#### 2.5.10.2.1 Monitoring before intervention

In the report of 2002, the ITAM stated that: "The gable wall of the church under consideration is most probably damaged due to the ageing process of its masonry, and has to be retrofitted either by restoration the mortar joints or by reinforcing the gable, e.g. by an additional supporting system inside the church garret.

#### 2.5.10.2.2 Monitoring and survey after intervention

Repair Measures Once all the works of retrofit were done, in 2007 the ITAM was asked again to measure the dynamic response of the structure, and specifically of the gable due to the ringing of the bells. The results of the measures of 2007, and there correspond value of 2002, are presented in the Table 2.14.

<table>
<thead>
<tr>
<th>Bell</th>
<th>Bell J1</th>
<th>Bell J2</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Year 2002</td>
<td>Year 2007</td>
</tr>
<tr>
<td></td>
<td>Point f</td>
<td>RMS f</td>
</tr>
<tr>
<td>12</td>
<td>1.32</td>
<td>0.15</td>
</tr>
<tr>
<td>20</td>
<td>1.32</td>
<td>0.11</td>
</tr>
<tr>
<td>2</td>
<td>1.32</td>
<td>0.08</td>
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</table>
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<tbody>
<tr>
<td>6</td>
<td>0.88</td>
<td>7.37</td>
<td>1.32</td>
<td>0.06</td>
<td>0.8</td>
<td>5.48</td>
</tr>
<tr>
<td>11</td>
<td>0.88</td>
<td>0.05</td>
<td>1.32</td>
<td>0.004</td>
<td>0.8</td>
<td>0.04</td>
</tr>
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</table>

2.5.11 Proposed monitoring

2.5.11.1 Dynamic monitoring
In order to recover the structural response of the structure, due to the dynamic excitation of the ringing of the bells, the dynamic response will be measured and analysed to observe the results of the previous retrofitting interventions.

2.5.12 Potential contribution to the project

2.5.12.1 Compliance with project objectives
The building offers opportunities for the testing and evaluation of the technologies and methods derived from the project.

The building can be considered for the application of new non-destructive technologies for material, mechanical and morphological characterization.

2.5.12.2 Potential contribution to validate the project’s methods and criteria
It is expected that the presented case study will contribute to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

2.5.13 Other specific opportunities provided by the proposed case study
The Church of St. Peter and Paul offers an interesting case regarding, in particular, the study of the performance and needs for protection of vulnerable buildings.

Table 2.15 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th></th>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection</td>
<td></td>
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<tr>
<td>(specify methods</td>
<td></td>
<td></td>
</tr>
<tr>
<td>and technologies)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric</td>
<td></td>
<td></td>
</tr>
<tr>
<td>survey</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>The material of the walls is masonry</td>
<td></td>
</tr>
<tr>
<td></td>
<td>mainly constituted by stone and lime</td>
<td></td>
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<td>2. Monitoring</td>
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2.1 Static monitoring

2.2. Dynamic monitoring
Monitoring in year 2002 and 2007

3. Structural analysis

3.1 Modelling and model updating
FEM modelling for elements (façades, typical bays, towers...)

3.2 Seismic analysis

4. Intervention

4.1 Characterization of needs for repair, maintenance and (seismic) strengthening
Repair and maintenance needs already defined.

4.1 Design of strengthening intervention
Steel plated were anchored in the gable and fixed to the structure of the roof.

4.2 Real implementation
Steel plates were anchored in the gable and fixed to the structure of the roof.

4.3 Monitoring during and after strengthening implementation
Implementation of a dynamic monitoring as part of the activities of the project.

2.5.14 References

- Beconcini M., Bennati S., Salvatore W. Structural characterisation of a medieval bell tower. Historical Constructions. Portugal, 2001
- Beconcini M, Croce P. Mengozzi, M., Dynamic Monitoring and Model Updating of a Masonry Bell Tower in Pisa. Structural Analysis of Historical Constructions, New Delhi, India 2006
2.6 MALLORCA CATHEDRAL, PALMA DE MALLORCA, SPAIN

2.6.1 Name, location and short description

The Cathedral of Saint Mary in the city of Palma, in the island of Mallorca, is a Gothic construction built during 13th to 15th c (Figure 2.179). The cathedral is classified as Cultural Heritage of National Interest (Spain).

Mallorca Cathedral is one of the most imposing medieval structures thanks to the immensity of its interior space and the extraordinary dimensions of its structural elements. The plan and the longitudinal section clearly show two distinct parts or bodies. The first body includes a central nave and two lateral ones, flanked as well by eight powerful abutments which lodge between the lateral chapels. The second body, built in a previous historical stage, includes the so-called Capella Reial (Royal Chapel), a single nave imposing Gothic construction by itself, and the smaller, but even older, Capella de la Trinitat (Trinity Chapel), the first element of the complex to be built. Remains of unused capitals and nervure springings at the end of the Royal Chapel suggest that the decision of building an imposing body of larger dimensions was actually taken after the completion of the eastern part of the building. The main body, built as a three-nave plan building with 8 large spans and lateral chapels between internally embedded buttresses. The diaphragmatic transverse arches of the naves are connected to a two-level, double battery of flying arches. The piers of the central nave, with octagonal cross section, are made of solid ashlar masonry with no rubble infill. The building is limited at the North by a cloister and a tower of square plan. In the past, the spandrels of the vaults were probably filled with pottery and lime mortar and were finished by an upper stone tile roof.

2.6.2 Historical note

The references available on the place and surrounding territory provide sufficient data to trace a historical sequence up to the present time. Before the cathedral, the location hosted other buildings erected by the Romans and the Muslims. Roman pedestals of possible statues possible belonging to a Roman forum have been found in the cloister of the cathedral. The existence of a Mosque before the construction of the Cathedral is very well documented as well, although no attempt has been made, for the moment, to uncover possible archeological remains. The presence of a reused paleo-Christian capital as part of a base of the present altar in the Real Chapel may be
demonstrating that the location constituted a religious place long before the Muslim conquest. It is well known that the location of the Arab Mosque is the same that is now occupied by the cathedral; in principle it seems that the cathedral was constructed around the Islamic building and that the latter was not destroyed until the Christian church was finished.

Comprehensive historical research has been already carried out mostly based on the analysis of ancient documents available in the files of the cathedral’s Chapter (Domenge 1997).

The construction began during the first reign of the insular dynasty, about year 1300, after king Jaume II (1276-1311) left an important legacy in his testament (1306) to support the cost of the Chapel of the Trinity. This first body had the mission of lodging the tombs of the royal family in its crypt. In the year 1311, the works of the following body of the apse began, namely the presbistery known as the Real Chapel. This phase concludes with the completion of the Real Chapel by 1370.

According to the most widely accepted interpretation, by 1330 it was decided to build the remaining construction according to a three-nave plan and yet, by the mid of 14th c., it was decided to increase the height of the vaults. The construction of the main nave developed during the rest of 14th and 15th c (with a major interruption from 1460 to 1570). The main façade, of noticeable Renaissance style, was built from 1594 to 1601, when the cathedral was consecrated.

Research on the historical books has provided significant hints on the construction process. The construction of the nave progressed, bay after bay, from the presbytery towards the façade (the last part to be built). Construction of the chapels was ahead because of the funding provided by noble families or corporations willing them as pantheons or gremial chapels (Domenge, 1997).

By the year the 1400 the works of the Cathedral concentrate in the door of the Mirador and by 1601, with a noticeable Renaissance style, the west, main façade is finished, with which the construction sites of the Cathedral are finalized.

During this phase, and according to Domenge (1997), an arc of the central nave fell in April of 1490 causing serious deterioration.

It has been possible, at least for one of the bays (the 4th one from the choir), to identify the process leading to its complete construction. It started with the lateral chapels, followed by the piers, then one lateral vault, then the other and finally the central one. In the case of this bay, the construction of the vaults lasted 7 years. It should be noted that during a period of about 5 years), the lateral vaults were already pushing against the pier while the lateral vault was not yet there to counteract their thrust.

The building has experienced significant problems and repairs. The 4th vault (previously discussed) partially collapsed 30 years after its construction. A significant number of vaults were repaired or reconstructed during the 17th, 18th and 19th centuries.

Due to the concerning out-of-plumb (about 1.3 m), the original façade was taken down replaced by a new one during the second half of 19th c. Demolition was decided in March 1851 and hence was not connected to the earthquake occurring in May the same year.

In the summer of 1851 the works of disassembling of the façade began under the direction of architect Antoni Sureda. In August of the following year architect Juan Peyronnet Baptist was committed with the design and the reconstruction of the new façade. The restoration project was presented in 1854. Preyroned laid-out a very different, flamboyant neo-Gothic façade completely strange to the neat design of the old façade and the rest of the building. The section of the buttresses of the new façade are significantly increased with respect of those of the former one. The works finished in 1888.

A series of intervention and restoration works have been undertaken during 20th c., among which the liturgical reform carried out by the Catalan architect Antoni Gaudi between 1904 and 1914 with the collaboration of other architects (Joan Rubió i Bellver, Josep M. Jujol, J. Towers Garcia and Guillem Reynés). Later, during the second half of 20th c. the existing timber roof over the high vaults was substituted by a new one composed of a light steel beams structure.

During last decades, the building has been subjected to continuous repair and maintenance works. The West façade and its towers have been very recently restored.
A more detailed and updated research, with more comprehensive information on the construction process and historical repairs, obtained after a close investigation on the historical files of the cathedral, can be found González and Roca (2003-2009).

2.6.3 Historical and artistic significance

Mallorca Cathedral is the main architectural historic monument in the Balearic Islands. It is also one of the largest and structurally more outstanding medieval buildings in Europe. Its construction involved a significant number of innovations allowing to improve and surpass the architectural and structural achievements brought by other Gothic structures.

In addition to its structural and architectural relevance, Mallorca cathedral includes a significant amount of valuable movable and immovable artworks produced from the Middle Age to modern times, as described in section 2.

2.6.4 Architectural arrangement, structure and materials

The length of the nave of the main body is of 77m and is distributed across seven bays. The width covered by the naves is of 35.3 m, of which 8.75x2 m are spanned by the lateral naves and 19.8 m by the central nave. The lateral naves are covered by pointed vaults of simple square plan; however, in the central nave they are of double square (rectangular) plan. This scheme is repeated both in all the bays of the naves except in the 5th one (from the choir), due to the presence of lateral doors (door of the Mirador and door of the Almoina) which provide it the role of a transept. In this bay, the longitudinal span of the vaults is slightly longer (Figure 2.180 to Figure 2.188).

The height reached by the vaults in their highest point (the key of the transverse arches) is of 43.95 m, surpassed solely with 44m by the Cathedral of Milan (with tied springings) and by the Cathedral of Beauveais with 46.30m. The Cathedral of Mallorca also is unique in being the Gothic cathedral with the highest lateral naves (29.4m). The octagonal piers have a circumscribed diameter of 1.6 and 1.7m and a height, of 22.7 at the level of the springings of the lateral vaults, thus attaining an overall slenderness ratio of 15. The slenderness of the piers, reaching a ratio of 14.6 between diameter and height, constitutes the more unique and audacious aspect of the building and contributes largely to a sense of internal great spaciousness.

The diaphanous interior space is made possible, in fact, by the very robust external buttressing system. The base of the main buttresses is 7.7 m long and 1.5 m wide; its maximum dimension represents a 44% of the span of the central nave. The lateral thrust of the high vaults is carried to the buttresses by a double battery of almost identical flying buttresses.

The building has never had a high pitched roof (causing lateral thrust due to wind pressure), but just a terrace over the vaults until the construction, in 18th c., of a traditional tile roof over timber beams, latter substituted by a more modern steel system.
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Figure 2.180 - View from exterior.

Figure 2.181 - View from interior.

Figure 2.182 - Detail of high vaults.
Figure 2.183 - Transverse section.

Figure 2.184 - Longitudinal section.

Figure 2.185 - Plan.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

Figure 2.186 - Double battery of flying arches.

Figure 2.187 - Dead weight over transverse arches.

Figure 2.188 - Dead weight over high vaults.
The transverse arches of both the lateral and central naves are diaphragmatic, meaning that they are provided with masonry wall spandrels filling all the space to the height of the key. The vaults are not filled or backed with rubble masonry, but just with a light structure composed of slender stone wallets and slabs. The structure of the cross section is complemented with significant overload, in the shape of triangular masonry masses placed upon the transverse arches and the keys of the vaults of the central nave. On the transverse arches a symmetrical triangular wall exists reaching its maximum depth at the key of the arch. On the keys of the vaults, the overload appears as a pyramid of square base (Figure 2.186 to Figure 2.188).

Recent historical research has provided light on the original quarries from which the stones where taken from. All of them are local quarries located at the sea-side allowing easy transportation of the material to the cathedral works by ship. Two main different types of stones were used to build the masonries of the cathedral. On one hand, most of the volume of the walls and buttresses is build by a bloc masonry composed of the so-called Marés stone, a well-known local type of limestone characterized by its very low strength and very easy workability. Laboratory mechanical tests have allowed the measurement of a low compression strength of only about 2 MPa. On the other hand, for the piers and flying arches another type of limestone of better quality was used, for which higher compression strength, about 20 MPa, has been measured.
The studies suggest that not only the external leaves of the walls and buttresses, but also their interior, is composed of large blocks of low strength of stone. Similarly, the piers are solid, their section being composed of a large square inner stone surrounded four pentagonal perimeter stones, all in the higher quality limestone.

2.6.5 Present and foreseen future use. People at risk

The cathedral has been used, since its construction, for the corresponding liturgical purposes. However, and since the popularization of the Balearic Islands as a touristic destination, the cathedral has become nowadays a powerful cultural attraction, hosting several million visitors each year. The island of Mallorca is visited each year by more than ten million tourists, of which the majority pay a visit to the cathedral. The building is visited by several thousand people each day, and may be hosting several hundred people at any moment during the visitor’s opening time. In addition, liturgy and cultural activities (such as concerts) are regularly celebrated.

2.6.6 Considerations on valuable cultural contents

Mallorca cathedral includes significant artistic contents including both movable and immovable works of art, such as sculptures, paintings and objects. The most remarkable works of art are (Figure 2.189 to Figure 2.191):

- Historical movable heritage such as a medieval timber choir, renaissance and baroque paintings and retabes and artistic ancient liturgical objects. Most of these artistic objects are nowadays hosted in the Cathedral museum located in several annex halls formerly used as sacristies and chapter houses (the so-called Gothic and Baroque chapter houses).
- Medieval stone sculptures and decoration in the main doors of the building (mostly in the Mirador’s (or South) door and the West façade’s door.
- In addition to the ancient works of art, the cathedral includes in its main nave decoration and several modernists objects, such as the pulpit and the balaquin model, designed by architect Antoni Gaudí and his cooperators (Josep Maria Jujol and Joan Rubió), at the beginning of 20th c.
• The artistic contents of the cathedral has been significantly enlarged in recent times with the contemporary art mural works by the modern artist Miquel Barceló in the so-called Saint Peter Chapel, done during 2001-2006.

2.6.7 Present condition and damage

2.6.7.1 Considerations on the present condition
In spite of some existing damage (see section 4.2), the structure is well preserved and no severe problem or active deterioration process has been identified. The structure has been subjected to continuous maintenance and repair since its construction, and some damage or partial collapses having occurred in the past were already subjected to repair in ancient times. Recent maintenance works have mostly involved the repair of cracks in walls, the stabilization and repair of the tracery of rose windows and the cleaning of façades.

2.6.7.2 Main observed alterations, damage and decay
The main existing structural damage is categorized as follows:

1- Cracking in piers. Cracks exist in a few piers Vertical or oblique cracks have developed across the stone extending, in some cases, to several rows (Figure 2.192). They tend to concentrate close to the corners of the octagonal section (the less confined parts) and, in some cases, shape full wedges partially or totally detached from the core of the pier. The reason for similar cracks in a few piers of Mallorca, randomly distributed, is less clear.

2- Cracking in walls and façade. Cracking, mostly developed along the mortar joints, can be also recognized in the exterior or clerestory walls (Figure 2.193). Part of such cracks is related to the out-of-plumbing experienced by the façade in the case of Mallorca Cathedral. Cracking is also observed in other structural components (as in buttresses, caused by existing openings or false windows between lateral chapels, and also in lateral vaults).

3- Cracking at vaults. The vaults of the central nave and the main transverse arches are separated by wide cracks developed throughout their contact lines (Figure 2.193).

4- Deformation. The deformation of the overall structure is perceptible. The piers show significant lateral deformation reaching, in some cases, up to 30 cm, i.e., 1/100 of height at the springing of the lateral vaults. Remarkably, both the magnitude and the shape of the deformation vary very significantly (almost randomly) among the different bays, or even between the two halves of a single bay Significant deformations are affecting the flying arches as, in special, those corresponding to the upper battery. Apparently, a few flying arches were, at some time, propped by means of masonry columns and walls to prevent their possible failure.

6- Leaning of western façade. The western façade seems to have experienced anew a certain out-of-plumb after its reconstruction, causing, in turn, some cracking in the panels of the clerestory wall close to it.

7 - Loss of mortar in mortar joints. This problem seems to have affected severely the structure in the past. Due to the disappearance of mortar, some of the vaults of the main nave had to be repaired or reconstructed during the early 18th. For the same reason, the flying buttresses had to be entirely repaired or substituted also during the 18th c. Continuous repointing of mortar joints has been carried out on exterior walls and flying arches also after their reconstruction. The reason for this loss of mortar is found in the invasion of the mortar by salts present in the stone. The large presence of salts in the stone is due to the fact that it was taken from quarries located at the seaside in order to allow ship transportation.
Figure 2.192 - Example of cracking in pier faces. Maps (left) and picture of parallel cracks close to a pier's coner.

Figure 2.193 - Examples of cracking in the clerestory wall (left, above), and between vault and transverse arch (right, above). Cracks in the vaults of the central nave (below).
2.6.8 Information on local seismicity

2.6.8.1 Local seismicity
The island of Mallorca is moderately seismic zone characterized by peak ground accelerations between 0.04g and 0.08g for a 475-year return period according to the European-Mediterranean Seismic Hazard Map of the European Seismological Commission (Jimenez et al., 2001) and only 0.04g according to the Spanish seismic code NCSE-02.

The Balearic Islands appear as rather calm place regarding seismicity. There is historical information on an earthquake having affected the city of Palma de Mallorca in 1660. Some authors (Fontseré, 1918) estimated for it a maximum intensity of VII in the MSK scale due to possible sources indicating the failure of one of two arches in the cathedral. It is now clear that these arches failed six months after the earthquake, which suggests that it was not direct reason for this collapse, while damage in these and other arches had been reported before (González and Roca 2004-2009). The true intensity of this earthquake may have been significantly lower. No record on possible effects of these earthquakes on the cathedral has been found in spite of intensive research. This might suggest that the effects have been normally minor or null.

The only earthquake known to affect the church was the one occurring in May 1851 (with estimated intensity between VII-VIII), causing damage to the façade and lateral towers. According to contemporary testimonies, no major damage appeared in the main structure of the building.

2.6.8.2 Characterization of the seismic action
The characterization of the seismic action for the Cathedral of Mallorca has been carried out in detail by Martínez (2008) as part of his Ph. D. dissertation. Deterministic and probabilistic earthquake hazard studies were undertaken to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building’s location.

2.6.9 Previous and on-going studies

2.6.9.1 Introduction
Previous detailed studies involving inspection, monitoring and structural analyses have been carried out since 2004 thanks to two different previous projects funded respectively by the Ministry of Culture of the Spanish Government (2004-2005 and 2007-2008) and the European Economic Cross Cultural Programme (contract ALA/95/23/2003/077-122, 2004-2006).

2.6.9.2 Inspection works

2.6.9.2.1 Geometric survey
A detailed photogrammetric survey of the entire building is available. Additionally, detailed survey of deformation has been carried out.

2.6.9.2.2 Research on materials
Microscopy and diffractometry were done on small amounts of both mortar and stone, sampled over the building to both identify the materials and their possible deterioration problems. A clear correlation was found between construction stages and different stone varieties. The fact that the construction was interrupted during almost a century, after the erection of the 4th bay (from the choir), is signalled by a change in the material used in the two phases. The substitution of all flying arches during 18th c. is also recognizable in that of them are built with the same variety of local sandstone, which, in turn, is not present in the rest of the building. Information has been also been gained on the extent of the repairs historical repairs (in particular, joint repointing).
2.6.9.2.3 Visual inspection
Detailed visual inspection has been carried out on the entire building (involving all walls, buttresses, vaults, arches and piers) for the purpose of identifying morphology, materials and damage. Cracking and damage has been mapped in detail.

2.6.9.2.4 Deep inspection by means of NDT and MDT
Deep inspection was carried out by means of the following technologies: (1) pulse radar, (2) seismic tomography and (3) dynamic testing.

Pulse radar was extensively used to obtain information on the internal morphology of the main structural members, including piers, walls and vaults. In the case of the piers, pulse radar, using a 1.5 GHz antenna, was applied in combination to seismic tomography to extend the results of the latter (corresponding only to a very limited sample) to the rest of the piers of the buildings. The walls and buttresses were inspected with a 900 Hz antenna. The radargrams show that the clerestory walls are composed of a single layer of about 45 cm wide stone blocks with no inner filling, while the buttresses include an infill of poorer material surrounded by an external stone block masonry leaf about 35 cm wide. Joints between stone blocks have low reflectivity indicating that there these are thin (with little mortar) across the external leaves and that there are no voids. A single core perforated in a buttress showed that the inner material consists of blocks of a poor and easy workable type of local limestone.

2D tomography was carried out on a sample of the piers in order to identify the internal morphology and the quality of the materials (Fig. 16). As is well known, seismic tomography consists of the 2D or 3D reconstruction of the propagation wave characteristics (speed and/or attenuation) through the material composing of natural material (soil) or structural members. Depending on the device used for the emission and the reception of the signal, tomography can be based on reflection, diffraction, refraction and transmission phenomena. Seismic tomography of transmitted waves was carried out by generating the signals with an instrumented iron head hammer with and collecting them with a high frequency and high sensitivity accelerometer. The acquisition was performed at 63500 samples per second. Post-processing, using a SIRT algorithm (Simultaneous Interactive Reconstruction Technique) produced the speed distributions shown in Fig. 16. Three nave piers of Mallorca Cathedral were selected to carry out detailed research. The theoretical coverage of the seismic tomography is shown in Fig. 16, left. The post-processing revealed that the section is made of 4 hexagonal stones surrounding a square inner one.

Dynamic characterization of Mallorca Cathedral was carried out by 18 ambient noise measurements done at different points of the structure during a windy day, using a 20s Lenartz seismometer. The location of the measurements was chosen so as to sample the entire building and included critical points on the extrados of the arches and vaults of the central nave, façade and buttresses. These measurements were carried out by a geophone with 120 dB dynamic range (from 10-3mm/s to 10-9mm/s) and lineal amplification (flat response) between 0.05 Hz and 40 Hz. In each point, the average autospectrum of 6 windows of 32 seconds each, scanned at 256 sps, was computed to obtain a frequency resolution of 1/32 Hz. Due to the great complexity of the ambient noise spectra, each temporal signal was filtered by an 8 point Butterworths band pass filter.
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NIKER Grant Agreement n° 244123

Figure 2.194 - Seismic tomography ray coverage (left) and resulting seed diagram for a pier (center). Resulting arrangement of pier (right).

About 10 modes could be identified with associated frequencies smaller than 4 Hz. Particle motion analysis was applied for each mode and point. Particle movement analyses permitted to identify qualitatively the shape of the modes by detecting the type of predominant deformation (transverse, longitudinal) and its distribution over the building.

2.6.9.2.5 Direct deep inspection
The nature of the filling over the vaults was inspected by the opening of some archaeological pits from the upper terraces. It turned out that the central vaults have no filling, the space between the vault membrane and the terrace being filled by means of small stone walls and slabs. The lateral vaults are filled with empty pottery vessels.

2.6.9.2.6 Other inspection works
The foundation soil was also investigated using a combination of geophysical techniques, including GRP, electrical tomography, refraction microtremor (ReMi) and measurements of the spectral ratio at 15 different points located both inside and outside the perimeter of the building.

2.6.9.3 Monitoring
Both dynamic and static monitoring were been recently implemented to the building.
Static monitoring included devices to measure a set of displacements at critical points of the buildings, clinometers, crackmeters, wind speed and orientation, temperature and humidity. It was implemented in 2004 and has been working until 2008.
A dynamic monitoring system was been implemented in Mallorca Cathedral, in April 2005, consisting of a 24-bit resolution dynamic acquisitor connected to two triaxial accelerometers, one of which has been installed on top of a vault of the central nave. The acquisitor’s clock is disciplined to GPS time by means of a GPS antenna. Continuous dynamic measurement is carried out to allow the capture of low-intensity oscillations. The system, with a sensitivity of $10^{-6}$g, allows continuous acquisition at 100 sps and 600-plus days of storage. Meaningful seismic episodes are detected thanks to information provided by the nearest seismological station ETO 8 in Mallorca. Information corresponding to any interval measured in GPS time can be then easily extracted from the entire data.
Unfortunately, no tremors with epicenter in the Balearic Islands, or close to them, have occurred during the period monitored. However, even very far earthquakes, as the Northern Chile earthquake of June 13, 2005, have produced measurable effects. The advantage of this sort of dynamic monitoring against the ambient vibration test described in section 3.6 becomes clear when the maximum amplitudes registered are considered. While the test permitted measurements of only $0.0015 \text{ m/s}^2$, the far epicentre earthquakes generated motions of the order of $0.01 \text{ m/s}^2$. In spite of the gain in one order of magnitude, these movements are still very small and it is expected that closer or local tremors, hopefully captured in the future, may provide more meaningful measurements.
Continuous dynamic monitoring has also shown that the frequencies of the structure experience a variation of about 10% over their mean value along the annual thermal cycle. This phenomenon is probably caused by the opening and closure of cracks due thermal volumetric changes.

2.6.9.4 Structural analysis

Previous results concerning dead loading and seismic performance of Mallorca Cathedral have been already obtained by different researchers (Maynou, 2001, Salas, 2002, Clemente, 2007, Clemente et al., 2006, Martinez et al, 2006, Martinez, 2008, Vacas, 2009) working in the frame of the Department of Construction Engineering of Technical University of Catalonia (UPC). These studies have been carried out using limit analysis (both static and kinematic) and finite element modeling.

These add to pioneering analysis carried out long time ago, such as the vector analysis by Rubio (1912) and the photoelasticity analyses by Mark (1984). Both studies have been considered into detail due to their important contribution to the understanding of the building.

Non-linear FEM detailed analyses of the typical nave bay have been carried out using a continuous damage constitutive model (Clemente et al. 2006). The material properties have been estimated based on laboratory tests on cores taken from the buildings or the original quarries. In order to take into consideration the construction process, a sequential analysis was carried out in which the changes experienced by the construction were subsequently simulated and adequately superposed. A tentative simulation of the construction process was undertaken on a model of the typical nave bay (Clemente, 2007) by means of a sequential analysis involving the following steps: (1) Construction of the lateral nave (buttresses and lateral vaults) and (2) construction of the central vault.

The study on the global structure, mostly oriented to seismic assessment, has been carried out on a global model prepared on purpose. However, and due to computer limitations, non-linear analysis up to failure was carried out on a series of macroelements representing distinct structural parts (Figure 2.196). These analyses allowed the determination of the capacity curves needed for the application of the capacity spectrum method.

Model updating and validation

The information provided by the non-destructive techniques was invested in preparing and validating, to the possible extent, the structural models. In these, the different members were modelled according to the results of the investigation: piers were defined solid, buttresses and walls were described as three-leaf or solid members, depending of each case, and the vaults were modelled with their existing infill. The latter aspect is of large importance, as the nature of the infill may have sensible influence on the overall mass of the vaults, which in turn may influence on the seismic response.

The structural model was updated using experimentally measured frequencies. To match the experimental frequencies, it was necessary to significantly increase the Young modulus in piers, vaults and buttresses with respect to some initially assumed values. The overall stiffness (value and distribution) was assessed by comparing experimental and numerical natural frequencies, the obtained from ambient vibration measurements (Martinez et al. 2006).
Figure 2.195 - Comparison of damage parameter for an instantaneous analysis (above, deformation x 300) and a sequential analysis involving two stages (below, deformation x 50).

Figure 2.196 - Global FEM model and some macroelements considered for the determination of the capacity curves (Martínez 2008).
2.6.10 Previous or on-going restoration works

2.6.10.1 General restoration works

Maintenance, involving the repair of cracks and other damage, is carried out in a continuous base. However, no consideration on the type of needed repairs, regarding the seismic performance of the building, has been so far undertaken in order to re-define and improve the maintenance practices.

Several parts of the structure have been recently subjected to restoration, mostly involving the cleaning of façades and consolidation of deteriorate masonry. This type of restoration has been applied to the tower and façades.

2.6.10.2 Actions oriented to seismic retrofitting

No actions specifically oriented to improve the seismic performance of the structure has been so far implemented. Based on the results of the studies more recently carried out, it has been proposed to undertake the following operations as a way to improve the seismic performance of the building:

Deep injection of cracks and local reconstruction (like for like replacement or “cuci-scuci”) of damaged masonry in walls and buttresses. In fact, this would involve the re-repair of many cracks which have been superficially repaired, in recent times, by means of shallow mortar repointing.

The implementation of a set of ties to improve the connection of the East façade to the rest of the building. According to the analysis recently carried out, some of the weakest foreseeable collapsing mechanisms are associated to the (insufficient) performance of the upper part of the East façade and its corresponding buttresses. More specifically, the proposed operation consists of the implementation of a couple of ties located tangent to the interior surface of the clerestory walls. The ties would be anchored, at one end, in the façade parament and, at the other end, on the clerestory wall itself after spanning a minimum of three bays.

As an alternative, the possibility of ties across the entire nave length (with anchorages at the West and East exterior paramounts, is also considered. In that case, the ties would be also useful to control the façade’s leaning. As revealed by the monitoring, the leaning of the façade is still developing at a very slow ratio. Constraining this trend is not considered prioritary (due to the slow ratio).

These proposals have been recently presented to the owners and have been accepted as possible actions to be undertaken in the near future depending on budget availability.

2.6.11 Historical research

Extensive historical research has been already undertaken within the frame of previous developed projects (see section 1). Additional historical investigation is still being carried out as part of the research developed by expert historians working on Mallorca cathedral (in particular, by historian Joan Domenge, who has been cooperating in the previous research projects.

2.6.12 Local seismicity and characterization of seismic action

Results on the characterization of the seismic action for Mallorca cathedral are already available thanks to previous specific research work by Martínez (2008). Although this information is considered sufficient for the purpose of the project, the possibility of carrying out additional investigation for a better characterization is not disregarded.

2.6.13 Proposed inspection works

2.6.13.1 Geometric survey. Aims, methods and technologies

As mentioned, a complete photogrammetric survey is already available. However, detailed laser-scanner survey of the interior and/or exterior is considered as an interesting possibility in order to
(1) enrich the available geometrical information and (2) for the purpose of comparing different
approaches and better identify their advantages and drawbacks.

2.6.13.2 Research on materials. Aims, methods and technologies
Non-destructive technologies resulting for the project will be applied for a better characterization of
the material physical and mechanical properties.

2.6.13.3 Deep inspection by means of NDT and MDT. Aims, methods and technologies
Dynamic survey (hammer and ambient vibration tests) of the columns of the Cathedral will be
undertaken by using an innovative non-contact radar interferometer,(IDS, mod. IBIS-S) recently
validated by Politecnico di Milano and available in the laboratories of Politecnico di Milano. The
radar technique has recently emerged as an innovative technology, suitable to the non-contact
vibration monitoring of large structures and the its application to the measurement of displacement
time-histories on historic and monumental structures was not yet developed in the literature and
has to be considered a great challenge.

2.6.14 Proposed monitoring

2.6.14.1 Static monitoring
A proposal has been prepared for the extension of the static monitoring undertaken during years
2004-2008 for an additional period of 4 years starting on 2011. The new monitoring system will
resort to wireless systems and will include crackmeters, inclinometers and base line displacement
transducers, in addition to the corresponding devices for the measurement of climatic parameters
(humidity, temperature and wind). The proposal has been presented to the owners (Chapter of the
Mallorca Cathedral). Its final approval is subject to budget availability and possibility for
sponsorship from local public institutions.

Given its significance, particular attention will be given to the monitoring of the internal temperature
in the building during the project duration. Temperature influences largely on the both static and
dynamic monitoring output, and a good characterization of its variation and distribution within the
building is necessary for a correct interpretation and post-processing of both results. Temperature
distribution will be recorded by means of a portable InfraRec G100 thermographic camera with
thermal sensibility of 0.08ºC allowing automatic image registration in time. This camera will be
working during the period corresponding, at least, to the duration of the project. This thermal
monitoring will be carried out, in any case, as part of the activities of the project (regardless of the
possibility of developing the above mentioned static monitoring proposal).

2.6.14.2 Dynamic monitoring
The dynamic monitoring will be carried out during the duration of the project by means of a system
including 3 triaxial high sensitivity accelerometers (Namometric’s Titan) connected to 3 high
capacity, high speed dynamic data acquisitors (Nanometric’s Trident 305). The system will be
disciplined to global time by means of a GPS antenna, allowing an accurate correlation between
the response measured in the building with seismic information obtained in the seismic stations
located in Mallorca island. The information can be obtained by means of a telematic system.

High density dynamic information will be continuously recorded. This continuous dynamic
monitoring has several connected purposes: (1) Characterize the dynamic response for ambient
vibration along with its dependence with environmental parameters (temperature, humidity); (2)
Capture the dynamic response in the occasion of microtremors of far epicenter earthquakes; (3)
Capture the dynamic response during intense wind episodes.

2.6.14.3 Monitoring phases
2.6.14.3.1 Monitoring before intervention
The static and dynamic monitoring systems proposed will be working before the possible execution of the proposed seismic strengthening solution. In fact, the new dynamic monitoring system was implemented on December 15, 2010. As an example, an accelerogram measured on December 19, 2010, capturing vibrations caused by a microtremor with epicenter in Catalonia is shown in Fig. 18.

2.6.14.3.2 Monitoring and control during intervention
Both the static and the dynamic system would be active during the execution of the proposed seismic intervention.

2.6.14.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step by step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the procedure applied.

Figure 2.197 - Accelerograms registered on December 19, 2010 in Mallorca cathedral with a triaxial accelerometer located over the high vaults. Vertical (top), transverse (center) and longitudinal vibration produced by a low intensity tremor with epicenter near Girona.

2.6.15 Proposed structural analysis. Aims and methods

2.6.15.1 Modeling and structural analysis methods
As a continuation of previous structural analyses already done in the recent past (see section 6.3), a global model of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment.

Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model) or larger parts of it (typical bays, façades, tower…). Appropriate constitutive equations for masonry based on damage theory or plasticity (with fragile models in tension) will be adopted.

2.6.15.2 Model updating and validation. Integration with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. In turn, numerical simulation using the models made available will be used for decision taking on the characteristics and general lay-out of static and dynamic monitoring.

2.6.15.3 Assessment of possible intervention methods

Structural models, using the same modeling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.6.16 Tentative proposed interventions

2.6.16.1 Conservation, repair and maintenance

The building is subjected to permanent maintenance according to traditional or conventional approaches. The required funds for on-going conservation and maintenance activity are granted by the significant contribution of the visitors to the Cathedral’s budget.

However, it is understood that the project should contribute with a more specific, scientifically based and global plan or proposal for the conservation and maintenance of the building, resulting from a better understanding of its structural performance and the application of the project’s resulting methods and criteria.

2.6.16.2 Need for seismic strengthening and proposed actions

As a result of the studies recently carried out, a repair and strengthening proposal has been already formulated. However, this proposal should be only regarded as a tentative proposal, to be modified and improved, and finally validated, based on the methods and criteria resulting from the project.

The initial proposal includes the following actions:

- Deep injection of cracks and local reconstruction (like for like replacement or “cuci-scuci”) of damaged masonry in walls and buttresses. In fact, this would involve the re-repair of many cracks which have been superficially repaired, in recent times, by means of shallow mortar repointing.

- The implementation of a set of ties to improve the connection of the East façade to the rest of the building. According to the analysis recently carried out, some of the weakest foreseeable collapsing mechanisms are associated to the (insufficient) performance of the upper part of the East façade and its corresponding buttresses. More specifically, the proposed operation consists of the implementation of a couple of ties located tangent to the interior surface of the clerestory walls. The ties would be anchored, at one end, in the façade parament and, at the other end, on the clerestory wall itself after spanning a minimum of three bays.

As an alternative, the possibility of ties across the entire nave length, with anchorages at the West and East exterior paraments, is also considered. In that case, the ties would be also useful to control the façade’s trend to increase its leaning. As revealed by the monitoring, the leaning of the façade is still developing at a very slow ratio. However, and due to the very slow ration, the leaning of the façade is not seen, at the moment, as a primordial issue.

These proposals have been already presented to the consideration of the owner (Chapter of Mallorca cathedral).

2.6.17 Potential contribution to the project

2.6.17.1 Compliance with project objectives

Mallorca cathedral provides an outstanding example of large and long-span heritage structure in need for seismic assessment and protection. Insufficient seismic capacity of Mallorca cathedral
would not only convey important risks on human life (as mentioned, the building is visited at any moment by hundreds of people), but might as well cause the loss of very valuable movable and immovable artistic heritage.

Although Mallorca Island is only subjected to moderate seismic risk, Mallorca cathedral constitutes in fact a rather vulnerable structure due to the large span and slenderness of its structural members and the lack of specifically seismic resistant structural devices or features. The building offers opportunities for the testing and evaluation of the technologies and methods derived from the project.

2.6.17.2 Foreseen opportunities for application of new approaches and technologies

The building can be considered for the application of new non-destructive technologies for material, mechanical and morphological characterization. The fact that it has already investigated recently by some available NDT methods offers interesting possibilities regarding the comparison and calibration of different methods. In particular, new methods can be tested and validated by comparing their prediction with already available information.

For instance, the output of techniques oriented to the survey of the interior of structural members (piers and walls) could be compared with the sectional restitutions already obtained by means of pulse radar and seismic tomography (see section 6.1.4). Similarly, new techniques for dynamic characterization can be also validated by comparison with the dynamic characterization made already available by more conventional ambient vibration testing by means of high-sensitivity accelerometers. This last possibility will be applied to a non-contact radar interferometer technology recently validated by Politecnico di Milano.

Significant effort will be carried out on the characterization of the structural response by means of both static and dynamic monitoring, and the use of the monitoring output for model updating and validation. Technologies for dynamic monitoring (as described in section 6.2) and for structural analysis will be tested and evaluated. Particular attention will be given to the combined use of monitoring and structural analysis through an integrated methodology allowing sound validation of structural models.

2.6.17.3 Foreseen opportunities for testing and validation of proposed intervention methods

As mentioned, a tentative intervention proposal, aimed at seismic improvement, has been already formulated. The intervention proposal will be reconsidered and improved by applying the methodologies and criteria resulting from the project. A better characterization of the actual strengthening needs is expected, leading to more refined decisions on strengthening approaches and methods. The possibility of alternative solutions, based on innovative technologies will be carefully examined. One of more alternative solutions, based on the project outcome, will be developed into detail for its possible real implementation.

The strengthening intervention may be actually implemented during the project duration (depending on several factors such as budget availability). In this case, the presented case study will also offer an interesting possibility for the evaluation and validation for the proposed intervention method.

2.6.17.4 Potential contribution to validate the project’s methods and criteria

The present case study can be considered for calibration of innovative measuring devices or systems for on-site application, oriented to characterize or measure both local (material) and, mostly, global (structural) properties and behaviour. Given the fact that Mallorca cathedral has been already investigated by means of some NDT techniques, the focus of further (as mentioned in section 6.1.4) should be placed on static and dynamic monitoring technologies oriented to model updating and dynamic characterization. The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.
2.6.17.5 Other specific opportunities provided by the proposed case study

Mallorca cathedral offers an interesting case regarding, in particular, the study of the performance and needs for protection of vulnerable buildings in moderate seismic regions. Mallorca cathedral is, to some extent, an extreme example (due to its large dimensions) of many ancient churches and palaces in the Iberian Peninsula and other Eastern Mediterranean regions which, in spite of being located in rather moderate seismic places, are however largely exposed to the effects of the earthquakes due to the lack of seismic risk awareness in local construction traditions and resulting seismically inappropriate structural types and features. In these regions, the large vulnerability of the structures causes significant risk levels (eventually, comparable to those of more prone seismic regions) in spite of the moderate seismicity.

Table 2.16- Summary of past, on-going and envisaged new activities

<table>
<thead>
<tr>
<th>Tasks already carried out or in course, with available results</th>
<th>Tasks to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>General photogrammetric geometric survey available.</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Stone properties identified based on samples taken from the building and the original quarries. Physical properties of mortar and stone, and mechanical properties of stone have been identified using conventional laboratory technologies on samples.</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT already used consisting of pulse radar, seismic tomography, dynamic testing. Dynamic survey (hammer and ambient vibration tests) of the columns of the Cathedral by using an innovative non-contact radar interferometer, (IDS, mod. IBIS-S) recently validated by Politecnico di Milano.</td>
</tr>
<tr>
<td>1.4 Other</td>
<td>Foundation soil has been characterized by geotechnical conventional and non-destructive geophysical tests.</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>5 years of continuous monitoring already available including temperature, humidity, wind parameters, base line extensometers, clinometers and crackmeters. Possibility to extend the static monitoring up to the end of the project using wireless devices Temperature distribution will be recorded by means of allowing automatic image registration in time. This camera will be continuously active during the duration of the project.</td>
</tr>
<tr>
<td>2.2. Dynamic monitoring</td>
<td>1 year of continuous monitoring available. Response for wind The dynamic monitoring will be improved with additional sensors</td>
</tr>
<tr>
<td>and several far-epicentre earthquakes captured.</td>
<td>(a total of 3 triaxial accelerometers) and will be kept active during the project duration.</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>3. Structural analysis</td>
<td></td>
</tr>
<tr>
<td>3.1 Modeling and model updating</td>
<td>Already available. FEM modelling for macroelements (façades, typical bays, transept) and global FEM models available. Model already updated according to dynamic monitoring results.</td>
</tr>
<tr>
<td></td>
<td>Models to be further improved and validated based on new monitoring results.</td>
</tr>
<tr>
<td>3.2 Seismic analysis</td>
<td>Done using FEM pushover calculations and kinematic limit analysis in combination with Capacity Spectrum Method.</td>
</tr>
<tr>
<td></td>
<td>To be improved using the new calculation criteria and methods derived from the project.</td>
</tr>
<tr>
<td>4. Intervention</td>
<td></td>
</tr>
<tr>
<td>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</td>
<td>Repair and maintenance needs already defined. Tentative characterization of seismic strengthening needs based on minimal intervention.</td>
</tr>
<tr>
<td></td>
<td>Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria.</td>
</tr>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td>Tentative design consisting of a system of ties improving the connection between the East façade and the lateral walls. To be developed in the form of a detailed project.</td>
</tr>
<tr>
<td></td>
<td>Should be reconsidered and improved based on the project new methodologies and criteria for seismic assessment. Possibility to apply incremental approach based on continuous monitoring</td>
</tr>
<tr>
<td>4.2 Real implementation</td>
<td>Strengthening proposal tentatively accepted by property.</td>
</tr>
<tr>
<td></td>
<td>The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Implementation of a long term dynamic monitoring as part of the activities of the project.</td>
</tr>
</tbody>
</table>

### 2.6.18 References

• Rubió, J. (1912) “Conferencia acerca de los conceptos orgánicos, mecánicos y constructivos de la Catedral de Mallorca”. Anuario de la Asociación de Arquitectos de Cataluña, Barcelona.
2.7 S. AGOSTINO CHURCH - L’AQUILA - ITALY

2.7.1 Name, location and short description

The church of St. Agostino is one of the best examples of Baroque architecture in L'Aquila and it is one of the most important eighteenth-century buildings with a central plan. The building was erected on the site of an older complex founded in 1282 by Carlo I d'Angiò, with the intervention of Bishop Niccolò Sinizzo. Around the mid-seventeenth century the building was enriched with new chapels and several decades later underwent a transformation style by Francesco Bedeschini. The earthquake of 1703 destroyed the church and the project of reconstruction was entrusted to the Roman architect Giovanni Battista Contini, who arrived in L'Aquila in 1707 to deal with the reconstruction of S. Bernardino. The intervention begun between 1709 and 1710, and after some interruptions in 1725 it was completed, except for the part and decorative furnishings, completed in subsequent years. The new church was conceived with an elliptical plan with the main entrance in correspondence to the major axis and a large apsidal presbytery on the opposite side. At the extremities of the minor axis two rectangular chapels were placed.

The main facade has two volumes: the upper one corresponding to the lantern and the lower one connected with the entrance.

The church was seriously damaged by the earthquake of the 6th of April 2009. The most relevant damages involved especially the drum, the dome and the lantern.

2.7.2 Geometrical, structural and material features

A preliminary geometric survey is available.
Figure 2.198 - Plan of the ground floor (left) and of the roof (right) of the church.

Figure 2.199 - Sections of the church.
2.7.3 Present and foreseen future use. People at risk.

The S. Silvestro church was severely damaged by the earthquake in 2009 and classified unfit for use since then. Nobody is allowed to enter the building due to the critical structural conditions of the internal structures.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterised by different coefficients of use (Table 2.27).

Its nominal life, according to the type of construction, is defined as $V_N \geq 50$ years. The reference period for the seismic action is given by the product $C_u \times V_N = 75$ years.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use $C_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The S. Silvestro church is classified in the 3rd class of use, with a coefficient of use $C_u = 1.5$.

2.7.4 Considerations on valuable cultural content

To be performed within the project.

The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:
Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
Categories of use (occasional or unused, frequent and very frequent)
The combination of these categories leads to define the exceeding probability (P) of the seismic action in 50 years and the importance factors γI for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.7.5 Information on local seismicity

2.7.5.1 Local seismicity
The city of l'Aquila is located in the central part of Italy, in the Abruzzo region. It is a high seismic zone characterized by a PGA (Peak Ground Acceleration) of 0.261g for a 475-year return period according to the Italian Code NTC 2008.
This region of central Apennines has one of the highest seismic hazard in Italy. Many destructive earthquakes are filed in the historical catalogues. The last dramatic event was the MW 6.7, 1915 Avezzano earthquake, located about 30 km to the south-west, that caused about thirty thousand deaths. Large earthquakes originate mainly on a narrow belt along the central Apennines, accommodating a NE-trending extension, with a rate of about 3 mm/yr, progressively thinning the Apennines thrust and fold belt.

2.7.5.2 Characterization of the seismic action during the 6th of April 2009 earthquake
On the 6th April 2009, a Mw 6.3 (MI 5.8) earthquake caused significant damage to L'Aquila, the medieval capital city of the Abruzzo region of Italy, and several surrounding towns and villages. 297 people were killed, 1,000 injured, 66,000 made homeless, and many thousands of buildings were destroyed or damaged.
With this seismic event the toll in terms of structural damage was enormous. In general, all the masonry buildings suffered a great amount of damage, in great part due to the fact that a vast amount of this buildings were made of poorly arranged masonry composed by round pebbles and mortar of scarce mechanical characteristics.
The L'Aquila earthquake presented a Richter Magnitude (MI) of 5.8, while strong-motion data inversion resulted in a moment magnitude (Mw) of 6.3 with a shallow focal depth, of approximately 8.0 to 9.0 km. The epicentre was located 10.0 km West of L'Aquila and 95.0 km NE of Rome. The main shock seemed to have been originated on a direct fault (the Paganica fault), 15.0 km long in NW-SE direction.
The earthquake involved a wide area among the cities of L'Aquila, Avezzano, Sulmona and Teramo. The ground morphology played an important role in the structural damage distribution and the most catastrophic effects were observed along the Aterno river valley.

2.7.6 Previous or on-going restoration works

2.7.6.1 Provisional strengthening intervention after the earthquake
Some provisional strengthening interventions have been already performed involving in particular the dome and the lantern of the church. During the project a detailed description of these interventions will be given.

2.7.7 Historical research
It will be necessary to carry out exhaustive historical investigations within the NIKER Project, focusing in particular on the effects of past interventions on the structural response of the structure during the earthquake.
2.7.8 Local seismicity and characterization of seismic action

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the building is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building's location.

2.7.9 Proposed (additional) inspection works

2.7.9.1 Geometric survey. Aims, methods and technologies

It is necessary to perform a complete and precise geometric survey of the building, using also new technologies developed during the project in order to enrich the available geometric information.

2.7.9.2 Deep inspection by means of NDT and MDT. Aims, methods and technologies

Within the NIKER project it will be possible to perform an extensive investigations’ campaign in order to characterize both quantitatively and qualitatively the masonry structures of the building and its global and local structural behaviour.

Deep inspections including both NDT and MDT are foreseen, such as: sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests,.. and other possible new investigation techniques derived from the project.

In particular the main objective is to concentrate the research activities on the dynamic survey of the building using ambient vibration tests. These investigations are necessary to design and install the proposed monitoring systems.

2.7.10 Proposed monitoring

2.7.10.1 Static monitoring

A static monitoring will be installed in the church during the project including devices to measure a set of displacements and strains at critical points of the structure.

The static monitoring system is composed by 15 channels:

8 displacements gauges installed on representative cracks in the middle part of the dome of the church to control the crack width.

4 displacement transducers to control the global displacements of the dome

3 thermal sensors to control both the air temperature and the walls temperature at different points of the structure

Data from the static system will be registered every 30 minutes.

Given its significance, particular attention will be given to the monitoring of the variation of temperature in the building during the project duration. Temperature influences largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the building is necessary for a correct interpretation and post-processing of both results.

The static monitoring will be kept active during the entire duration of the project.

2.7.10.2 Dynamic monitoring

The proposed dynamic monitoring is composed by 18 high sensitivity piezoelectric accelerometers connected to an acquisition unit with a Wi-Fi router for remote data transmission. Three reference sensors will be fixed at the base of the structure to record the ground acceleration both in operational conditions and during seismic events. Three accelerometers will be placed on the lantern to store the structural response (amplification) of the church, whereas the other sensors will be installed around the perimeter of the dome’s base according to the results of the structural dynamic identification.
High-density (100 sps) dynamic information will be continuously recorded. This continuous dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for ambient vibration along with its dependence with environmental parameters (temperature, humidity); (2) Capture the dynamic response in the occasion of possible seismic events.

2.7.10.3 Monitoring phases

2.7.10.3.1 Monitoring before intervention
The static and dynamic monitoring systems will be working before the execution of the strengthening interventions and restoration works that are going to be performed soon.

2.7.10.3.2 Monitoring and control during intervention
Both the static and the dynamic system will be kept active during the execution of the proposed interventions.

2.7.10.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step-by-step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.7.11 Proposed structural analysis. Aims and methods

2.7.11.1 Modeling and structural analysis methods
It will be necessary to create and implement FE models of the church and to calibrate them on the results of investigations and monitoring in order to perform advanced linear and non-linear FE structural analysis.

Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model). Local analysis will be implemented by means of the kinematic limit analysis method.

2.7.11.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general layout of static and dynamic monitoring.

2.7.11.3 Assessment of possible intervention methods
Structural models, using the same modeling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.7.12 Envisaged interventions

2.7.12.1 Considerations on the need for conservation, repair and maintenance
The building has been heavily damaged by the earthquake and urgently needs the design and implementation of strengthening interventions and seismic retrofitting in order to stop the damage evolution.

2.7.12.2 Need for seismic strengthening and proposed actions
The most suitable strengthening interventions involving the entire restoration and/or reconstruction of the collapsed parts of the church will be designed and implemented during the project.

2.7.13 Potential contribution to the project

2.7.13.1 Compliance with project objectives
The proposed case study gives the opportunity to:
Prove the reliability of the adopted models for the seismic assessment and the structural analysis
Verify the effectiveness of the proposed structural solutions.
Evaluate the application of new technologies and methods developed during the project.

2.7.13.2 Foreseen opportunities for application of new approaches and technologies
The building can be considered for the application of new non-destructive technologies for material, mechanical and morphological characterization.
Significant effort will be carried out on the characterization of the structural response by means of both static and dynamic monitoring, and the use of the monitoring output for model updating and validation. Technologies for dynamic monitoring (as described in section 6.2) and for structural analysis will be tested and evaluated. Particular attention will be given to the combined use of monitoring and structural analysis through an integrated methodology allowing sound validation of structural models.

2.7.13.3 Foreseen opportunities for testing and validation of proposed intervention methods
A tentative intervention proposal, aimed at the restoration, reconstruction and seismic improvement, will be formulated during the project. The intervention proposal will take into account the application of methodologies and criteria resulting from the project. The possibility of alternative solutions, based on innovative technologies will be carefully examined. One of more alternative solutions, based on the project outcome, will be developed into detail for its possible real implementation.

The strengthening intervention may be actually implemented during the project duration (depending on several factors). In this case, the presented case study will also offer an interesting possibility for the evaluation and validation for the proposed intervention method.

2.7.13.4 Potential contribution to validate the project’s methods and criteria
The focus of further researches should be placed on the implementation of NDT and MDT tests for the characterization of the material and structure properties and on static and dynamic monitoring technologies oriented to model updating and dynamic characterization.
The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

2.7.13.5 Other specific opportunities provided by the proposed case study
The case study of the church of S. Agostino offers an interesting case regarding the study of the performance, the seismic assessment and the design of strengthening interventions in a building strongly damaged by an earthquake in a post seismic emergency scenario.
It could become a pilot project for the implementation of a standardized procedure in the reconstruction process of the building belonging to cultural heritage in the l'Aquila city centre and in the surrounding historical centres hit by the earthquake.
Table 2.18 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>Complete and precise geometric survey of the building</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Not expected within the project</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT consisting of sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests. The research effort will concentrate in particular on the dynamic survey of the building.</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>A continuous static monitoring system is currently being installed including temperature and humidity sensors, linear displacement gauges on the main cracks and dome displacement transducers.</td>
</tr>
<tr>
<td>2.2. Dynamic monitoring</td>
<td>A continuous dynamic monitoring system is currently being installed including eighteen high sensitivity piezoelectric accelerometers.</td>
</tr>
<tr>
<td>3. Structural analysis</td>
<td></td>
</tr>
<tr>
<td>3.1 Modeling and model updating</td>
<td>FE models creation and implementation. Need to calibrate the models on the results of investigations and monitoring. Model updating techniques exploiting the monitoring results.</td>
</tr>
<tr>
<td>3.2 Seismic analysis</td>
<td>To be performed using FEM pushover calculations and kinematic limit analysis in combination with Capacity Spectrum Method and the new calculation criteria and methods derived from the project.</td>
</tr>
<tr>
<td>4. Intervention</td>
<td></td>
</tr>
<tr>
<td>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</td>
<td>Tentative characterization of seismic strengthening needs based on minimal intervention. Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria.</td>
</tr>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td>Design of strengthening interventions and seismic retrofitting to be performed during the project.</td>
</tr>
<tr>
<td>4.2 Real implementation</td>
<td>The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td>Possibility to apply incremental approach based on continuous monitoring. Available for monitoring during and after</td>
</tr>
</tbody>
</table>
2.7.14 References

- Guidelines for evaluation and mitigation of seismic risk to cultural heritage with reference to technical constructions regulation, Ministry for Cultural Heritage and Activities, General Direction of Architectural Heritage and Landscape - Dept. of Civil Protection Agency (Italy)
- Modello A-DC / B-DP PCM-DPC MiBAC 2006 - Scheda per il rilievo del danno ai beni culturali - Chiese e Palazzi. source: www.protezionecivile.it
2.8 THE S.MARCO CHURCH - L’AQUILA - ITALY

2.8.1 Name, location and description

The St. Mark church is located in the L’Aquila city centre. The first construction of the church dates back to the end of the 13th century - beginning of the 14th century. The church was built by initiative of the habitants of Pianola, a small town in the outskirts of L’Aquila, and it is located on the hearth of the city centre, between Via dei Neri and Piazza della Prefettura. Medieval traces are preserved mainly in the external walls and in the lateral portal, which dates back to the 14th century. The main façade was likely built at the beginning of the XV century. The building was completely restructured around 1750, after the 1703 devastating earthquake. The two bell towers belong to that period.

It was severely damaged by the 6th of April 2009 earthquake. Even if the present-day external appearance of the church seems to indicate an almost untouched structure, the inner view of the church clearly reveal the seriousness of the situation. After the earthquake the church reported severe damage in the apsidal and transept area, where a critical crack pattern was noticed in the external walls, which manifested a visible outward overturning, involving the four pillars sustaining the dome. Also the transversal response of the church proved to be inadequate, since the most part of the vaults collapsed, such as a big portion of the external wall, at the clerestory level. Severe damage was finally reported in the vaults of the apse, of the presbytery, in the triumphal arch.

2.8.2 Historical note

The San Marco church is one of the first churches built in L’Aquila in the second half of 13th century, whose construction was carried out due to the initiative of Pianola’s inhabitants. The church is located on the hearth of the city of L’Aquila, between “Via dei Neri” and “Piazza della Prefettura”. The medieval traces are preserved mainly in the external walls and in the lateral entrance, dated from the 14th century. The main façade could have been built at the beginning of the XV century.

The entrance on the left side is considered more ancient than the one on the main façade. The minor entrance has to be grouped with the other of identical setting present in S. Antonio, in Santa Maria del Guasto, in San Vito, in the Madonna del Carmine, etc... that presents identical structural and decorative components. In San Marco church there are four symbols of the Evangelists, three
to the left and one to right, that adorn the lintel with the usual "Agnus Dei". The decoration is completed with the figures of St Abate and of San Marco's insignia.

In the last century the internal late-baroque covering, coming from the 1700s, has been restored. The medieval structures added on the facade on the XVIII century have not changed anything of the original beauty. This beauty is increased by the presence of the gothic single ancient windows, and by the "Aquilano" texture presents in the masonry external leaf, [1] and [10].

2.8.2.1 Past transformations and interventions

The church as it is nowadays presents several signs that indicate numerous transformations. The most important were: (a) the partial reconstruction of the church in 1315 after the earthquake; (b) the lateral chapels built in the XVI century covered with stone vaults, attached to the nave lateral walls; (c) the complete modification of the building in 1750 with the construction of the two bell towers and of the top part of the frontal façade, Figure 2.201.

Throughout the years the San Marco church underwent several structural and non-structural maintenance works, Figure 2.202. The main interventions were performed in 1970, 2005 and 2007. In 1970, the church was subjected to a very intrusive intervention that consisted on the removal of the entire pre existent timber roof and its replacement with a new one made by prefabricated beams and roof slab. In order to make this roof as self supporting as possible steel ties were placed on each of the alignments of the pre-fabricated beams. During the intervention of substitution of the roof it was also constructed over the presbytery a R.C. structure to support the new roof it in this area. In 2005 different maintenance works were performed, as the replacement of the old iron ties positioned on the top part of the bell towers by more recent steel ties with a more efficient fixing mechanism (Figure 2.202) In 2007 the arches that support the dome over the presbytery area were reinforced using carbon fiber layers fixed to the arches inner face (Figure 2.202).

![Figure 2.201 - View of San Marco church before the earthquake.](image1)

![Figure 2.202 - Main interventions on San Marco church.](image2)
2.8.3 Geometrical, structural and material features

The building is here described, through the (1) geometrical survey of the structure based on the measurements attained and on the geometrical info given by the L'Aquila C.H. architect; (2) detailed description of each of the composing constructive elements; (3) analyses of the material that composes the different structural elements of the church, based on a 1st level wall quality form “Survey of masonry typology and masonry quality”, [15].

2.8.3.1 Geometrical Survey

A simple geometrical survey, Figure 2.203, of the church was performed based on the available elements such as topographic surveys, which were later validated during the technical inspections to the building, through control measurements.

![Geometrical characterization of San Marco Church](image)

Figure 2.203 - Geometrical characterization of San Marco Church. (a) Main façade. (b) Right façade.
2.8.3.2 Description of the Structural Elements

The original timber trusses of the roof were substituted by R.C. elements in the second half of the last century with large clay elements between them.

A shallow dome with a slightly elliptical plan is covering the central part of the transept. The dome is supported by 4 arches made in a solid brick masonry which continue in concentric circles at the spring of the dome, while the central part of the dome is made with brick headers. At the extrados of the arches wooden tie beams are inserted in the masonry as confinement. The lateral filling of the dome is made with rather regular stones well distributed. From the analysis of the collapsed materials the dimension of the bricks was measured as 290x150x30mm.

The barrel vault covering the nave is constituted by a mixed system: six arches in brick masonry divide five spans made with reed, (Figure 2.204a).

The thickness of the arches is probably 290mm and they are similar to the arches supporting the dome. Also in their case timber tie rods are present at the extrados inserted in the masonry. The lateral filling of the barrel vault is made with mixed brick and stone masonry regularly positioned.

The vaults of the aisles are made with mixed brick and stone masonry. The lateral filling is made with loose pieces of bricks and stones without any mortar. The semi dome of the apse is badly damaged and it was probably made with a mixed brick and stone masonry, (Figure 2.204b).

All the examined load-bearing walls are made by a multiple leaf masonry with no connection between the leaves. All the pictures show a high non homogeneity of the masonry due to different construction phases and modifications probably after past earthquakes, (Figure 2.204c).

The base of the walls and the left lateral wall are made externally by irregularly cut stone, while the right lateral wall and the facade are externally made with regularly cut stones.

2.8.3.3 Material Quality

The survey of the different types of masonry present in the church was performed during the technical inspections to the church. To systematize the collected information, a specific form developed at Politecnico di Milano (POLIMI) was used (1st level wall quality form “Survey of masonry typology and masonry quality”), [15], for each type of masonry present in the structure. The survey sites, (Figure 2.205), were chosen taking into consideration the historical information and consequently the different constructive phases.

In some cases the walls are composed by more than one type of masonry, as so, and according to its composition a weighted average of the properties as to be considered, especially in what concerns the use of this properties in simplified analysis.

Further surveys, especially to the walls section, weren’t possible due to the poor safety conditions presented by the church, with possibility of localized collapses.
2.8.4 Present and foreseen future use. People at risk.

The S. Marco church was severely damaged by the earthquake in 2009 and classified unfit for use since then. Nobody is allowed to enter the church due to the critical structural conditions of the internal structures of the building.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterised by different coefficients of use (Table 2.27).

Table 2.19 - Classes and coefficients of use $C_U$ for buildings.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use $C_U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums,...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The S. Marco church is classified in the 3rd class of use, with a coefficient of use $C_U = 1.5$.

The nominal life, according to the type of construction, is defined as $V_N \geq 50$ years. The reference period for the seismic action is given by the product $C_u \times V_N = 75$ years.

2.8.5 Consideration on valuable cultural contents

The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:

- Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
- Categories of use (occasional or unused, frequent and very frequent)

The combination of these categories leads to define the exceeding probability ($P$) of the seismic action in 50 years and the importance factors $\gamma_I$ for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).
2.8.6 Present condition and damage

2.8.6.1 Considerations on the present condition
The 6th of April earthquake induced severe damage on the church. The calculated damage index, according to the form for the damage survey of C.H. - Churches [7], corresponded to the value of 0.66 (on a scale from 0 to 1, where 0 is related to total absence of damage, and 1 to total collapse of the building), which is a relatively high value when compared to the average attained values for the other churches in L’Aquila. Different activated mechanisms of the San Marco church surveyed were analyzed in detail and a set of possible causes for its activation were hypothesized [14].

2.8.6.2 Main observed damage and decay
The several technical inspections to the church during the months following the earthquake allowed the creation of detailed internal and external crack pattern maps, (Figure 2.206). The interpretation of the crack pattern can be of great help in understanding the state of damage of the structure, its possible causes and the type of survey to be performed, provided that the development history of the building is already known.

In this case the crack pattern maps allowed an easy and intuitive assessment of the damage during the seismic event. The overall damage of the church is very high. The most severe damages are: the total collapse of the apse vault, (Figure 2.207a); a wall portion located on the upper part of the nave left lateral façade, (Figure 2.207b); part of the chapels adjacent to this façade, (Figure 2.207c); some of the arches that support the nave vault and also approximately 80.0% of the nave vault, (Figure 2.207d).

It is also worth noticing the separation between the internal and external masonry of the nave right lateral façade, probably due to the lack of interlock, as they belonged to two different constructive phases, Figure 2.208a. The dome over the presbytery presents an advanced crack pattern map, Figure 2.208b. The supporting structure is compromised: one of the arches that support this dome collapsed, while the other three are heavily damaged. As for the main façade, it presents an out-of-plane deformation characterized by the presence of vertical cracks in correspondence with intersections between the main façade and the nave lateral walls. This façade presents also important in-plane shear damage passing through the entire thickness of the wall, Figure 2.208d.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

2.8.7 Information on local seismicity

2.8.7.1 Local seismicity

The city of L'Aquila is located in the central part of Italy, in the Abruzzo region. It is a high seismic zone characterized by a PGA (Peak Ground Acceleration) of 0.261g for a 475-year return period according to the Italian Code NTC 2008 [13].

This region of central Apennines has one of the highest seismic hazard in Italy. Many destructive earthquakes are filed in the historical catalogues. The last dramatic event was the MW 6.7, 1915 Avezzano earthquake, located about 30 km to the south-west, that caused about thirty thousand deaths. Large earthquakes originate mainly on a narrow belt along the central Apennines, accommodating a NE-trending extension, with a rate of about 3 mm/yr, progressively thinning the Apennines thrust and fold belt.

2.8.7.2 Characterization of the seismic action during the 6th of April 2009 earthquake

On the 6th April 2009, a Mw 6.3 (MI 5.8) earthquake caused significant damage to L'Aquila, the medieval capital city of the Abruzzo region of Italy, and several surrounding towns and villages.

297 people were killed, 1,000 injured, 66,000 made homeless, and many thousands of buildings were destroyed or damaged.

With this seismic event the toll in terms of structural damage was enormous. In general, all the masonry buildings suffered a great amount of damage, in great part due to the fact that a vast amount of this buildings were made of poorly arranged masonry composed by round pebbles and mortar of scarce mechanical characteristics.

The L'Aquila earthquake presented a Richter Magnitude (MI) of 5.8, while strong-motion data inversion resulted in a moment magnitude (Mw) of 6.3 with a shallow focal depth, of approximately 8.0 to 9.0km. The epicentre was located 10.0 km West of L'Aquila and 95.0 km NE of Rome. The main shock seemed to have been originated on a direct fault (the Paganica fault), 15.0 km long in NW-SE direction.

Figure 2.207 - Collapse damages on the church.

Figure 2.208 - Activated mechanisms on the church.
The earthquake involved a wide area among the cities of L’Aquila, Avezzano, Sulmona and Teramo. The ground morphology played an important role in the structural damage distribution and the most catastrophic effects were observed along the Aterno river valley.

2.8.8 Previous and on-going studies

2.8.8.1 Introduction
Inspection works already carried out are rather limited and involved only the external structure of the church due to the heavy state of damage of the inner part of the building. Before the execution of any kind of inspections and investigations it will be necessary to perform some provisional interventions to guarantee adequate safety conditions.

2.8.8.2 Inspection works

2.8.8.2.1 Geometric survey
A geometric survey of the entire building is available. Additionally, structural and damage survey have been carried out.

2.8.8.3 Monitoring
In parallel to the execution of the first provisional interventions on the external part of the church, the opening of the main cracks was controlled by means of an automated low-cost structural monitoring system [5].

The monitoring system is continuously acquiring data, and stores hourly the readouts coming from 5 linear displacement transducers positioned in the external area of both apse and transept, where the worst damage scenario is observed (Figure 6). Data are correlated to the environmental parameters recorded by a temperature - relative humidity sensor positioned at the base of the scaffolding.

A couple of acceleration sensors is located at the base of the structure, in order to record any relevant data in terms of seismic events - also of low-moderate energy - and other two sensors are positioned at the top of the North wing of the transept, to store the structural response (amplification) of the church. The system is able to automatically store the data when the acceleration in one of the sensors exceeds a predefined threshold, both in time and frequency domain, and to periodically record the data (e.g. at a 12-24 hrs intervals) with the aim of the repetition of the dynamic identification, to measure possible variations in the modal parameters of the structure with the progression of the strengthening interventions. Sensors, both at the base and at the top, are positioned orthogonally to each other, measuring accelerations in the horizontal plane. The monitoring system was installed on the 10th of August 2009.

Figure 2.209 - Automatic Monitoring System installed on the San Marco church.
2.8.8.4 Structural analysis
A global FE model of the church is already available. A dead weight analysis and a natural frequency analysis have been already performed. More detailed and thorough analysis will be carried out during the project duration.

2.8.9 Previous and on-going restoration works

2.8.9.1 Provisional strengthening intervention after the earthquake
In the post-seismic scenario the emergency activities to protect the buildings belonging to cultural heritage have been developed at two parallel levels: (i) damage survey and design, (ii) implementation of temporary safety measures.

The design of temporary interventions for the safety of an historical building starts from the damage survey and from the identification of the collapse mechanisms activated from the seismic action.

The initial provisional strengthening intervention was financed by the Veneto Region of Italy, which paid 240.000,00 € as a first installment. Works started the 4th of July 2009 and were completed in November of the same year. This first intervention aimed at counteracting the most critical collapse mechanisms, such as the apsidal and transept walls overturning. Further interventions involved the hooping of the facade, the sustaining of a wide walls portion of the right longitudinal side of the church, and the strengthening - by means of wooden trusses - of the valuable stone main portal.

These interventions were the first and the most urgent in emergency time, and were carried out by working outside the church - considered the serious risk of collapse of further elements of the structure, especially in the period of continuation of seismic events (aftershocks) with non negligible magnitude.

The initial intervention entailed the construction of a sound scaffolding made by hollow pipe steel trusses (Figure 4), constituting a portal spanning across the church, in order to inhibit the prosecution of the overturning of the apses and transepts walls. Two lattice towers were built on the two sides of the church, subsequently connected at their top by an open web girder, in its turn connected to the masonry structures of the church.
In the façade area, wooden struts were employed in order to counteract the overturning of a remarkable part of the right wall external veneer, and to sustain the stone elements of the valuable church portal, disconnected by the seismic motion. The façade was hooped on two levels with steel cables, also positioned to avoid the shear damage progression. Finally, several openings were propped by means of wooden trusses, and the façade bell-towers were hooped at the level of the belfry.

The continuation of the works (2010-2012), with further installments provided by the Veneto Region for a total of 5 Millions of Euros, will entail the “internal” works - also considered that the aftershocks are today substantially of negligible magnitude. Works will consist in the removal (with sorting of valuable and reusable materials) of the internal debris, the construction of a lattice steel tower sustaining the breach in the wall on ‘Prefettura square’, and then in the main internal provisional intervention, consisting in the creation of steel “portals” in lattice structure, used to prop the still unstable masonry vertical elements, mainly in order to avoid the risk of flexural collapse at mid height of the pillars. This metallic hollow pipe structure will be designed in a way to be the base for the definitive strengthening interventions and local reconstruction of masonry elements.

2.8.10 Historical research

It will be necessary to carry out additional historical investigations within the NIKER Project, focusing in particular on the effects of past interventions on the structural response of the structure during the earthquake.

2.8.11 Local seismicity and characterization of seismic action

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the church is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building’s location.
2.8.12 Proposed (additional) inspection works

2.8.12.1 Geometric survey. Aims, methods and technologies
A complete and rather precise geometric survey of the building is already available. However, a more detailed survey, using also new technologies is considered as an interesting possibility in order to enrich the available geometric information.

2.8.12.2 Deep inspection by means of NDT and MDT. Aims, methods and technologies
Once installed the provisional structure in the church it will be possible to perform an extensive investigations' campaign in order to characterize both quantitatively and qualitatively the masonry structures of the building.
Deep inspections include NDT and MDT such as: sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests, and other possible new investigation techniques derived from the project.

2.8.13 Proposed monitoring.

2.8.13.1 Static monitoring
The static monitoring already installed on the S. Marco church will be kept active during the project duration.
Given its significance, particular attention will be given to the monitoring of the variation of temperature in the building during the project duration. Temperature influences largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the building is necessary for a correct interpretation and post-processing of both results.

2.8.13.2 Dynamic monitoring
The dynamic monitoring already installed in the fortress will be kept active during the project duration.
High-density dynamic information will be continuously recorded. This continuous dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for ambient vibration along with its dependence with environmental parameters (temperature, humidity); (2) Capture the dynamic response in the occasion of possible seismic events.

2.8.13.3 Monitoring phases

2.8.13.3.1 Monitoring before intervention
The static and dynamic monitoring systems will be working before the execution of the provisional strengthening intervention in the inner part of the church and then also before the restoration works and the implementation of the seismic strengthening solutions.

2.8.13.3.2 Monitoring and control during intervention
Both the static and the dynamic system will be active during the execution of the proposed interventions.

2.8.13.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step by step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.8.14 Proposed structural analysis. Aims and methods
2.8.14.1 Modeling and structural analysis methods
It will be necessary to calibrate the available FE models on the results of investigations and monitoring in order to perform advanced linear and non-linear FE structural analysis. Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macro modelling approaches involving the entire structure (global model) or larger parts of it. Local analysis will be implemented by means of the kinematic limit analysis method.

2.8.14.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general lay-out of static and dynamic monitoring.

2.8.14.3 Assessment of possible intervention methods
Structural models, using the same modeling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.8.15 Envisaged interventions

2.8.15.1 Considerations on the need for conservation, repair and maintenance
The building has been heavily damaged by the earthquake and urgently needs the design and implementation of strengthening interventions and seismic retrofitting in order to stop the damage evolution.

2.8.15.2 Need for seismic strengthening and proposed actions
The most suitable strengthening interventions involving the entire restoration and/or reconstruction of the collapsed parts of the church will be designed and implemented during the project duration.

2.8.16 Potential contribution to the project

2.8.16.1 Compliance with project objectives
The proposed case study gives the opportunity to:
- Prove the reliability of the adopted models for the seismic assessment and the structural analysis
- Verify the effectiveness of the proposed structural solutions.
- Evaluate the application of new technologies and methods developed during the project.

2.8.16.2 Foreseen opportunities for application of new approaches and technologies
This case study can be considered for the application of a knowledge-based methodology developed during the project, which consists in the implementation of a step-by-step procedure or incremental approach. The interventions techniques or the intervention scheme must allow for progressive decision-making and implementation of intervention measures. The final decision is substantially supported by the progressively increasing knowledge of the real response of the building acquired before, during and after any step, by appropriately studying the real case in its real environmental conditions and conditions of use.

Another important aspect concerns the definition of guidelines and criteria for the reconstruction of the damaged parts of the building, indicating the most appropriate strengthening interventions on vertical and horizontal elements according to the techniques tested in the project.
2.8.16.3 Foreseen opportunities for testing and validation of proposed intervention methods

A tentative intervention proposal, aimed at the restoration, reconstruction and seismic improvement, will be formulated during the project. The intervention proposal will take into account the application of methodologies and criteria resulting from the project. The possibility of alternative solutions, based on innovative technologies will be carefully examined. One of more alternative solutions, based on the project outcome, will be developed into detail for its possible real implementation.

The strengthening intervention may be actually implemented during the project duration (depending on several factors). In this case, the presented case study will also offer an interesting possibility for the evaluation and validation for the proposed intervention method.

2.8.16.4 Potential contribution to validate the project’s methods and criteria

The focus of further researches should be placed on the implementation of NDT and MDT tests for the characterization of the material and structure properties and on static and dynamic monitoring technologies oriented to model updating and dynamic characterization.

The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

2.8.16.5 Other specific opportunities provided by the proposed case study

The case study of the S. Marco church offers an interesting case regarding the study of the performance, the seismic assessment and the design of strengthening interventions in a building strongly damaged by an earthquake in a post seismic emergency scenario.

It could become a pilot project for the implementation of a standardized procedure in the reconstruction process of the building belonging to cultural heritage in the L’Aquila city centre and in the surrounding historical centers hit by the earthquake.

Table 2.20 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>1- Inspection (specify methods and technologies)</th>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 Geometric survey</td>
<td>General geometric survey available</td>
<td></td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Not available</td>
<td>No direct inspection works are foreseen at the moment.</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT consisting of sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests</td>
<td></td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>1 year of continuous static monitoring already available including temperature and humidity sensors and five linear displacement transducers</td>
<td>The static monitoring will be kept active during the project duration.</td>
</tr>
<tr>
<td>2.2. Dynamic monitoring</td>
<td>1 year of continuous dynamic monitoring available constituted by four high sensitivity piezoelectric accelerometers</td>
<td>The dynamic monitoring will be kept active during the project duration.</td>
</tr>
</tbody>
</table>
### 3. Structural analysis

<table>
<thead>
<tr>
<th>Subsection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1 Modeling and model updating</td>
<td>FE models available. Need to calibrate the models on the results of investigations and monitoring.</td>
</tr>
<tr>
<td>3.2 Seismic analysis</td>
<td>To be performed using FEM pushover calculations and kinematic limit analysis in combination with Capacity Spectrum Method and the new calculation criteria and methods derived from the project.</td>
</tr>
</tbody>
</table>

### 4. Intervention

<table>
<thead>
<tr>
<th>Subsection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</td>
<td>Tentative characterization of seismic strengthening needs based on minimal intervention. Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria.</td>
</tr>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td>To be developed in the form of a detailed project.</td>
</tr>
<tr>
<td>4.2 Real implementation</td>
<td>The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td>Possibility to apply incremental approach based on continuous monitoring. Available for monitoring during and after execution of strengthening.</td>
</tr>
</tbody>
</table>

### 2.8.17 References

- Modello A-DC / B-DP PCM-DPC MiBAC 2006 - Scheda per il rilievo del danno ai beni culturali - Chiese e Palazzi. source: www.protezionecivile.it


http://portale.ingv.it/portale_ingv

www.reluis.it


http://www.stru.polimi.it:8180/index.jsp
2.9 S. SILVESTRO CHURCH - L’AQUILA - ITALY

2.9.1 Name, location and short description

The church of St. Silvestro is located in the city centre of L’Aquila. It was built in Romanesque-Gothic style during the first half of the 14th century. A series of 15th century frescoes in the apse, executed by an unknown artist, which represents scene from the life of Christ, a Pietà, Saints and Prophets makes it most interesting. The facade, with a beautiful Romanesque door-way and its characteristic rose-window, dominates a quiet piazza where the Branconio Palace also stands. The restoration of the interior, carried out in the late 1960’s revealed the original antique building with ogival arches. The Branconio Chapel to the left of the main altar is the place where the famous painting “The Visitation” by Raffaello Sanzio, donated by the artist to his Aquilan friend, Giambattista Branconio, was originally placed.

The church was seriously damaged by the earthquake of the 6th of April 2009. The most relevant damages involved especially the bell tower, the facade and the three apses.

2.9.2 Geometrical, structural and material features

A preliminary geometric survey is available.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

Figure 2.212 - S. Silvestro church: plan view.

Figure 2.213 - Present condition of the church and its bell tower after the earthquake.
2.9.3 Present and foreseen future use. People at risk.

The S. Silvestro church was severely damaged by the earthquake in 2009 and classified unfit for use since then. Nobody is allowed to enter the building due to the critical structural conditions of the internal structures.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterised by different coefficients of use (Table 2.27).

Table 2.21 - Classes and coefficients of use C_U for buildings.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use C_U</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
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<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The S. Silvestro church is classified in the 3rd class of use, with a coefficient of use C_U = 1.5.
The nominal life, according to the type of construction, is defined as V_N ≥ 50 years. The reference period for the seismic action is given by the product C_u x V_N = 75 years.

2.9.4 Considerations on valuable cultural content

The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:
Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
Categories of use (occasional or unused, frequent and very frequent)
The combination of these categories leads to define the exceeding probability (P) of the seismic action in 50 years and the importance factors γ_i for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.9.5 Information on local seismicity

2.9.5.1 Local seismicity

The city of l’Aquila is located in the central part of Italy, in the Abruzzo region. It is a high seismic zone characterized by a PGA (Peak Ground Acceleration) of 0.261g for a 475-year return period according to the Italian Code NTC 2008.

This region of central Apennines has one of the highest seismic hazard in Italy. Many destructive earthquakes are filed in the historical catalogues. The last dramatic event was the MW 6.7, 1915 Avezzano earthquake, located about 30 km to the south-west, that caused about thirty thousand deaths. Large earthquakes originate mainly on a narrow belt along the central Apennines, accommodating a NE-trending extension, with a rate of about 3 mm/yr, progressively thinning the Apennines thrust and fold belt.
2.9.5.2 Characterization of the seismic action during the 6th of April 2009 earthquake

On the 6th April 2009, a Mw 6.3 (MI 5.8) earthquake caused significant damage to L’Aquila, the medieval capital city of the Abruzzo region of Italy, and several surrounding towns and villages. 297 people were killed, 1,000 injured, 66,000 made homeless, and many thousands of buildings were destroyed or damaged.

With this seismic event the toll in terms of structural damage was enormous. In general, all the masonry buildings suffered a great amount of damage, in great part due to the fact that a vast amount of this buildings were made of poorly arranged masonry composed by round pebbles and mortar of scarce mechanical characteristics.

The L’Aquila earthquake presented a Richter Magnitude (MI) of 5.8, while strong-motion data inversion resulted in a moment magnitude (Mw) of 6.3 with a shallow focal depth, of approximately 8.0 to 9.0 km. The epicentre was located 10.0 km West of L’Aquila and 95.0 km NE of Rome. The main shock seemed to have been originated on a direct fault (the Paganica fault), 15.0 km long in NW-SE direction.

The earthquake involved a wide area among the cities of L’Aquila, Avezzano, Sulmona and Teramo. The ground morphology played an important role in the structural damage distribution and the most catastrophic effects were observed along the Aterno river valley.

2.9.6 Previous and on-going studies

2.9.6.1 Inspection works

2.9.6.1.1 Geometric survey

A rather precise geometric survey of the entire building is available.

2.9.7 Previous or on-going restoration works

2.9.7.1 Provisional strengthening intervention after the earthquake

Some provisional strengthening interventions have been already performed involving the bell tower, the main façade and the apse of the church. During the project a detailed description of these interventions will be given.

2.9.8 Historical research

It will be necessary to carry out exhaustive historical investigations within the NIKER Project, focusing in particular on the effects of past interventions on the structural response of the structure during the earthquake.

2.9.9 Local seismicity and characterization of seismic action

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the building is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building's location.

2.9.10 Proposed (additional) inspection works

2.9.10.1 Geometric survey. Aims, methods and technologies

It is necessary to perform a complete and precise geometric survey of the building, using also new technologies developed during the project in order to enrich the available geometric information.
2.9.10.2 Deep inspection by means of NDT and MDT. Aims, methods and technologies
Within the NIKER project it will be possible to perform an extensive investigations’ campaign in
order to characterize both quantitatively and qualitatively the masonry structures of the building
and its global and local structural behavior.
Deep inspections including both NDT and MDT are foreseen, such as: sonic pulse velocity tests,
single and double flat jack tests, dynamic identification tests... and other possible new investigation
techniques derived from the project.
In particular the main objective is to concentrate the research activities on the dynamic survey of
the building using ambient vibration tests. These investigations are necessary to design and install
the proposed monitoring systems.

2.9.11 Proposed monitoring

2.9.11.1 Static monitoring
A static monitoring will be installed in the bell tower of the church during the project including
devices to measure a set of displacements and strains at critical points of the structure.
The static monitoring system is composed by 13 channels:
2 inclinometers to control the displacement of the bell tower and the main façade in the two in-
plane directions (2 x 2 = 4 channels).
7 displacements transducers installed on representative cracks in the lower and in the middle part
of the bell tower to control the crack width.
2 thermal sensors to control both the air temperature and the walls temperature at different points
of the structure
Data from the static system will be registered every 30 minutes.
Given its significance, particular attention will be given to the monitoring of the variation of
temperature in the building during the project duration. Temperature influences largely both the
static and dynamic monitoring output, and a good characterization of its variation and distribution
within the building is necessary for a correct interpretation and post-processing of both results.
The static monitoring will be kept active during the entire duration of the project.

2.9.11.2 Dynamic monitoring
The proposed dynamic monitoring is composed by 6 high sensitivity piezoelectric accelerometers
connected to an acquisition unit with a Wi-Fi router for remote data transmission. Three reference
sensors are fixed at the base of the structure to record the ground acceleration both in operational
conditions and during seismic events. The positioning of the other acceleration sensors on tower’s
elevation will be decided on the results of the structural dynamic identification.
High-density (100 sps) dynamic information will be continuously recorded. This continuous
dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for
ambient vibration along with its dependence with environmental parameters (temperature,
humidity); (2) Capture the dynamic response in the occasion of possible seismic events.

2.9.11.3 Monitoring phases

2.9.11.3.1 Monitoring before intervention
The static and dynamic monitoring systems will be working before the execution of the
strengthening interventions and restoration works that are going to be performed soon.

2.9.11.3.2 Monitoring and control during intervention
Both the static and the dynamic system will be kept active during the execution of the proposed
interventions.
2.9.11.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step-by-step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.9.12 Proposed structural analysis. Aims and methods

2.9.12.1 Modeling and structural analysis methods
It will be necessary to create and implement FE models of the bell tower and the church and to calibrate them on the results of investigations and monitoring in order to perform advanced linear and non-linear FE structural analysis.

Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model). Local analysis will be implemented by means of the kinematic limit analysis method.

2.9.12.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general layout of static and dynamic monitoring.

2.9.12.3 Assessment of possible intervention methods
Structural models, using the same modeling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.9.13 Envisaged interventions

2.9.13.1 Considerations on the need for conservation, repair and maintenance
The building has been heavily damaged by the earthquake and urgently needs the design and implementation of strengthening interventions and seismic retrofitting in order to stop the damage evolution.

2.9.13.2 Need for seismic strengthening and proposed actions
The most suitable strengthening interventions involving the entire restoration and/or reconstruction of the collapsed parts of the bell tower and the church will be designed and implemented during the project.

2.9.14 Potential contribution to the project

2.9.14.1 Compliance with project objectives
The proposed case study gives the opportunity to:
- Prove the reliability of the adopted models for the seismic assessment and the structural analysis
- Verify the effectiveness of the proposed structural solutions.
- Evaluate the application of new technologies and methods developed during the project.
2.9.14.2 Foreseen opportunities for application of new approaches and technologies

The building can be considered for the application of new non-destructive technologies for material, mechanical and morphological characterization.

Significant effort will be carried out on the characterization of the structural response by means of both static and dynamic monitoring, and the use of the monitoring output for model updating and validation. Technologies for dynamic monitoring (as described in section 6.2) and for structural analysis will be tested and evaluated. Particular attention will be given to the combined use of monitoring and structural analysis through an integrated methodology allowing sound validation of structural models.

2.9.14.3 Foreseen opportunities for testing and validation of proposed intervention methods

A tentative intervention proposal, aimed at the restoration, reconstruction and seismic improvement, will be formulated during the project. The intervention proposal will take into account the application of methodologies and criteria resulting from the project. The possibility of alternative solutions, based on innovative technologies will be carefully examined. One of more alternative solutions, based on the project outcome, will be developed into detail for its possible real implementation.

The strengthening intervention may be actually implemented during the project duration (depending on several factors). In this case, the presented case study will also offer an interesting possibility for the evaluation and validation for the proposed intervention method.

2.9.14.4 Potential contribution to validate the project’s methods and criteria

The focus of further researches should be placed on the implementation of NDT and MDT tests for the characterization of the material and structure properties and on static and dynamic monitoring technologies oriented to model updating and dynamic characterization.

The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

2.9.14.5 Other specific opportunities provided by the proposed case study

The case study of the church of S. Silvestro offers an interesting case regarding the study of the performance, the seismic assessment and the design of strengthening interventions in a building strongly damaged by an earthquake in a post seismic emergency scenario.

It could become a pilot project for the implementation of a standardized procedure in the reconstruction process of the building belonging to cultural heritage in the l’Aquila city centre and in the surrounding historical centers hit by the earthquake.

Table 2.22 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>Complete and precise geometric survey of the building</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Not expected within the project</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT consisting of sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests. The research</td>
</tr>
</tbody>
</table>
2. Monitoring

2.1 Static monitoring
A continuous static monitoring system is currently being installed including temperature and humidity sensors, linear displacement transducers on the main cracks and two inclinometers.

2.2. Dynamic monitoring
A continuous dynamic monitoring system is currently being installed including six high sensitivity piezoelectric accelerometers.

3. Structural analysis

3.1 Modeling and model updating
FE models creation and implementation. Need to calibrate the models on the results of investigations and monitoring. Model updating techniques exploiting the monitoring results.

3.2 Seismic analysis
To be performed using FEM pushover calculations and kinematic limit analysis in combination with Capacity Spectrum Method and the new calculation criteria and methods derived from the project.

4. Intervention

4.1 Characterization of needs for repair, maintenance and (seismic) strengthening
Tentative characterization of seismic strengthening needs based on minimal intervention. Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria.

4.1 Design of strengthening intervention
Design of strengthening interventions and seismic retrofitting to be performed during the project.

4.2 Real implementation
The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.

4.3 Monitoring during and after strengthening implementation
Possibility to apply incremental approach based on continuous monitoring. Available for monitoring during and after execution of strengthening.
2.9.15 References

- Guidelines for evaluation and mitigation of seismic risk to cultural heritage with reference to technical constructions regulation, Ministry for Cultural Heritage and Activities, General Direction of Architectural Heritage and Landscape - Dept. of Civil Protection Agency (Italy)
- Modello A-DC / B-DP PCM-DPC MiBAC 2006 - Scheda per il rilievo del danno ai beni culturali - Chiese e Palazzi. source: www.protezionecivile.it
2.10 THE SPANISH FORTRESS - L’AQUILA - ITALY

2.10.1 Name, location and description

The Spanish fortress is located in the city centre of l’Aquila (Abruzzo region - Italy) and it is one of the most impressive Renaissance castles in Central and Southern Italy. In the 15th century L’Aquila became the second most powerful city in the Kingdom of Naples, under the Spanish domination. In 1528 Viceroy Filiberto d’Orange ordered to build a fortress in the highest North spot of the city, according to the project of a famous Spanish architect, Don Pirro Aloisio Escrivà. The construction started in 1534; Escrivà designed a giant fortress, composed by four bastions connected through heavy walls, 60 meters long, with a thickness of 30 m at the bottom and 5 m at the top. All around the fortress there was a ditch (never filled with water) 23 meters wide and 14 meters deep, aimed at defending the foundations from the enemy artillery. The Fortress, which was built not to defend the city, but to control it (many cannons pointed to the city) and to be a completely self-sufficient structure, was never used in battles. Its cannons, always ready to fire, were silent throughout the centuries: the only victim was the city itself, whose decline began with the construction of the fortress and went on under the Spanish domination. Between 1949 and 1951 the castle was restored, and chosen as the seat of the National Museum of Abruzzo.

2.10.2 Historical note

Il Castello Spagnolo dell’Aquila (The Spanish Castle of L’Aquila) is one of the most notable and best-preserved accomplishments of the military architecture on Italian modern land. The constructional events and the same architectural features of the huge fortress make up an exemplary historical testimony of the turbulent years of the horrible wars of Italy, a period of profound and dramatic transformations, in the political structure of the peninsula as well as in the art of war and fortifications while substantial part of Italian territory fell under the direct Spanish dominations, the internal political and social equilibrium changed deeply at various conditions and the traditional spaces were drastically reduced in the city’s autonomy; others significant changes were verified in the wartime technological field, with the use of the cannonball artillery. It was finally forced to overcome the secular medieval typology of the fortifications. The Castle made up one of the principal strongholds of the reinforcement plans of the reign of Naples carried out by the Viceroy don Pedro de Toledo in 1532. The enormous expense for the construction was inexcusably charged to the city, to punish the population for the revolt which burst out on the 31st December 1528 and which was suppressed in the following February by Filiberto d’Orange’s troops. In 1534 Toledo granted Perro Luis Escrivà, the captain of the imperial army, the commission to design and manage the work. Even if he had not had the possibility to carry out any important work up till then, he already had a reputation as a “great architect, and very skilled in...
fortifications”. He was born into a nobile family from Valencia in about 1490. His work experience, which began in Spain during the revolt of the comuneros, was enriched during the recent conflict between Carlo V and the Lega di Cognac: from the Apologia we come to learn in fact, that in 1528 he participated in the defence of Naples against the siege of Lautrec. The commission, which was granted by the Viceroy Toledo, offered him the opportunity to put into practice his knowledge of the Arts, philosophy and geometric-mathematics no less than the acquisitions of his long military training and castle building. The designing task could have been carried out in complete freedom, without being conditioned by the existing constructions which were to be incorporated or reutilized in the new fortress, nor did the characteristics of the land impose unavoidable solutions.

![Figure 2.214 - New edition of the Perspective Plan of Fonticulano (1600) Engraved by Bleau, Published by Mortier in Amsterdam in 1680.](image)

In the period in which it was built, il Castello Spagnolo dell’Aquila “the Spanish Castle of L’Aquila” constituted the most advanced result of the new military architecture. Thanks to the sacrifices of the city, which was forced to take on most of the buildings expenses, the modern science of the fortifications and the aesthetic plannings of the designer were carried out in the most accomplished way, with a rare coherence and rationality. The technical solutions adopted by Escrivà are the result of the conscious prefiguration of the potential wartime by a possible invading army and a careful evaluation of all the possibilities offered by the modern siege strategies. He tried to oppose an efficient defense device to every offence threat. Escrivà personally ran the work up to 1535; he then returned to Naples and had the opportunity to meet the Emperor who had come to visit the city after the victory of Tunis. The work was carried out for the most part over a twenty year period, even if only in 1567 the city was relieved of its heavy taxes which were imposed on it for the construction. In that year the first part of the construction phase ended. During the same year the part of the fortification linked to the military was carried out. It was made up of the four powerful bastions on a pentagonal plan, the four strong barrages, the moat and an external earthwork. The elegant double order arcade originally foreseen for the entire length of the perimeter of the internal courtyard was instead limited only to the southeastern wing.
2.10.3 Structural and material features

The castle belongs to the square-shaped fortresses with pentagonal bastions at the four corners. A massive wall ten meters thick at the foundation level composes the external structure. The fortress is protected by a ditch 23 meters wide and 14 deep. The sides of the corner bastions are 35 meters, while the building itself is 60 meters long. The bastions are 28 meters high and are composed by two levels with vaulted ceiling with an average height of 10 meters. The four wings of the fortress create an inner courtyard, from which it is possible to access to the different sections of the building.

Figure 2.215 - Plan view of the fortress.
Figure 2.216 From the top to the bottom: a) South-East elevation; b) South-West elevation; c) North-East elevation

Figure 2.217 - Sections of the building.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

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Figure 2.218 - The main entrance of the fortress.

Figure 2.219 - External view in the pre-seismic scenario.
2.10.3.1 Masonry

The thick defensive walls are made of stone with elements of unequal size, partly hewn with an external surface of travertine. The internal bearing walls are made of stone with an irregular arrangement, whereas the pillars of the arcade on the southeast side of the building, are made of large square stone of travertine. The thickness of the internal walls is different in each wings of the fortress: between 60 and 75 cm for the walls of the north west side; between 80 and 90 cm for the south west side; between 50 and 80 cm for the north east one; between 80 and 114 cm for the south east part.
2.10.3.2 Floors and roof
In the floor at the level of the courtyard there are brick vaults in every room, alternating between barrel vaults and cross vaults. On the upper levels different floor typologies are present: concrete and masonry slabs, steel beams with hollow flooring block or brick vaults.

The original timber roof was substituted during past interventions with a heavy reinforced concrete roof. In most cases a false ceiling is present made of steel beams and hollow flooring block.

2.10.4 Present and foreseen future use. People at risk.

The Spanish fortress has been used as a museum since the early 50’s and as the seat of the Department of National Heritage and Cultural Activities for the Abruzzo Region. It was severely damaged by the earthquake in 2009 and classified unfit for use since then. Nobody is allowed to enter the castle apart from the workers involved in the restoration works.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterised by different coefficients of use (Table 2.27).

Table 2.23 - Classes and coefficients of use CU for buildings.

<table>
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<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use CU</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
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<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The Spanish fortress is classified in the 3rd class of use, with a coefficient of use $C_U = 1.5$. 
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

The nominal life, according to the type of construction, is defined as \( V_N \geq 50 \) years. The reference period for the seismic action is given by the product \( C_u \times V_N = 75 \) years.

2.10.5 Considerations on valuable cultural contents

Between 1949 and 1951 the castle was restored, and chosen as the seat of the Museo Nazionale d'Abruzzo. The entrance is through a bridge in stone, passing under a portal with the giant coat-of-arms of Emperor Charles V. The Museum is on three floors: on the ground floor, there is the giant skeleton of an Elephas Meridionalis (improperly called mammoth, scientific name Archidiskon) found a few miles from Aquila in 1954, and an archeological section with pieces of the Italic, pre-Roman period, a section with inscriptions and pieces from the Roman towns in Abruzzo, among them a fine Roman calendar from Amierno (25 AD).

On the first floor the medieval and modern art section, with works of Abruzzese artists of the centuries 13th to 17th such as: the Polyptich by Jacobello del Fiore; a Processional Cross by Nicola from Guardiagrele, a group of wooden and terracotta sculptures such as St. Sebastian work of Silvestro dell'Aquila; then paintings by Flemish and Roman and Neapolitan artists such as Conca, Bedeschini, Solimena, De Mura; finally the contemporary art section with such artists as M. Vaccari, R. Guttuso, V. Guidì, G. Capogrossi, O. Tamburi, R. Brindisi.

Other valuable artistic contents of the Spanish fortress are:

- The stone portal
- The decorated and painted wooden ceilings of the two representative rooms of the Governor's residence
- The prisons
- The venerable parish of Santa Barbara in the “Regio Castello”
- The mummies

The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:

Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
Categories of use (occasional or unused, frequent and very frequent)

The combination of these categories leads to define the exceeding probability (P) of the seismic action in 50 years and the importance factors \( \gamma_I \) for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.10.6 Present condition and damage

2.10.6.1 Considerations on the present condition

The Spanish fortress was seriously damaged by the earthquake of the 6th of April 2009. The most relevant damages and collapses involved especially the upper floors of the Fortress. According to the damage survey template for palaces used in the technical inspections (Model B-DP PCM-DPC MiBAC 2006), overturning and flexural mechanisms on the external walls, shear damage in the external and internal walls, damages to vaults and arches, local collapses of floors and vaults, correspond to the most worrying observations. The damages shown by the building were remarkable both for intensity and distribution, and were considered so serious to be likely menacing the overall stability of some large parts of the building.

2.10.6.2 Main observed damage and decay

The South-East wing (Figure 2.223) of the fortress underwent the most significant structural problems. On the external front it can be noted the overturning of the upper masonry walls; several
collapses took place at the second floor together with a considerable separation of the floors from the longitudinal walls.

In the internal front of the same side the pillars of the porch arcade show crushing failure mechanisms. The seismic event damaged the walls of the first floor and produced a longitudinal crack in the barrel vault of the arcade. The crack is also visible on the upper floor, meaning that it interests the entire thickness of the slab. There are also many shear cracks on the transverse bearing walls (Figure 2.224).
In the South-West wing it can be noted an overturning mechanism of the two facades, cracks on vaults and shear cracks on the transverse walls. Moreover, on the first and second floor the separation between external perimeter walls and internal walls and floors are clearly evident (Figure 2.225).

The two facade of the North-West part showed a greater resistance to the overturning mechanism; there are no large detachments of the floors from the perimeter walls. In fact in this area of the fortress a system of tie rods connecting the perimeter walls had been inserted before the earthquake. This intervention was effective and avoided collapses and irreversible damages to the structure (Figure 2.226). However, the transverse walls are seriously damaged and on the second floor some parts of masonry walls and floor slabs collapsed.

The North-East wing is affected by a slight overturning mechanism of the two facades. There are some shear cracks on masonry walls but the most evident damages are localized on the upper floors.

2.10.7 Information on local seismicity

2.10.7.1 Local seismicity

The city of l’Aquila is located in the central part of Italy, in the Abruzzo region. It is a high seismic zone characterized by a PGA (Peak Ground Acceleration) of 0.261g for a 475-year return period according to the Italian Code NTC 2008 (Norme Tecniche per le Costruzioni DM 14-01-2008). This region of central Apennines has one of the highest seismic hazard in Italy. Many destructive earthquakes are filed in the historical catalogues. The last dramatic event was the MW 6.7, 1915 Avezzano earthquake, located about 30 km to the south-west, that caused about thirty thousand deaths. Large earthquakes originate mainly on a narrow belt along the central Apennines,
accommodating a NE-trending extension, with a rate of about 3 mm/yr, progressively thinning the Apennines thrust and fold belt.

2.10.7.2 Characterization of the seismic action during the 6th of April 2009 earthquake

On the 6th April 2009, a Mw 6.3 (MI 5.8) earthquake caused significant damage to L’Aquila, the medieval capital city of the Abruzzo region of Italy, and several surrounding towns and villages. 297 people were killed, 1,000 injured, 66,000 made homeless, and many thousands of buildings were destroyed or damaged.

With this seismic event the toll in terms of structural damage was enormous. In general, all the masonry buildings suffered a great amount of damage, in great part due to the fact that a vast amount of this buildings were made of poorly arranged masonry composed by round pebbles and mortar of scarce mechanical characteristics.

The L’Aquila earthquake presented a Richter Magnitude (Ml) of 5.8, while strong-motion data inversion resulted in a moment magnitude (Mw) of 6.3 with a shallow focal depth, of approximately 8.0 to 9.0km. The epicentre was located 10.0 km West of L’Aquila and 95.0 km NE of Rome. The main shock seemed to have been originated on a direct fault (the Paganica fault), 15.0 km long in NW-SE direction.

The earthquake involved a wide area among the cities of L’Aquila, Avezzano, Sulmona and Teramo. The ground morphology played an important role in the structural damage distribution and the most catastrophic effects were observed along the Aterno river valley.

2.10.8 Previous and on-going studies

2.10.8.1 Introduction

Previous detailed studies involving inspections and monitoring activities have been carried out immediately after the earthquake starting from July 2009.

2.10.8.2 Inspection works

2.10.8.2.1 Geometric survey

A detailed geometric survey of the entire building is available. Additionally, detailed structural and damage survey have been carried out.

2.10.8.2.2 Visual inspection

Detailed visual inspection has been carried out on the entire building (involving all walls, buttresses, vaults, arches and piers) for the purpose of identifying morphology, materials and damage. Cracking and damage has been mapped in detail.

2.10.8.2.3 Deep inspection by means of NDT and MDT

A series of non destructive and minor destructive testing were performed in the Spanish Fortress (Figure 2.250) by the research groups of the Politecnico of Milan and the University of Padova in order to characterize and evaluate the state of damage of the masonry structures (walls and pillars) quantitatively and qualitatively and to identify the structural response of the most damaged wings of the palace (especially the South-East one).

- The experimental campaigns included:
  - Sonic pulse velocity tests
  - Radar tests
  - Thermographic tests
  - Single and double flat jack tests
  - Dynamic identification tests
The ND tests were mainly aimed at understanding the reasons for the high damage of the heavy pillars and of the transversal walls at the ground floor.

![Plan of the ground floor of the castle](image)

**Figure 2.227 - Plan of the Spanish fortress: position of the tests at the ground floor of the SE wing.**

### 2.10.8.2.3.1 Sonic tests

Three sonic tomographies were performed. The measurement of sonic pulse velocities are combined along different ray-paths on a cross section of masonry and on two pillars, and are subsequently processed in order to define mean values of velocity on each portion of the wall section itself.

Two pillars located on the inner side of the South-East wing and a vertical section of a thick masonry wall in the South-West area were tested.

### 2.10.8.2.3.2 Radar tests

On one pillar previously investigated by means of sonic tests, radar tests were carried out using a high frequency antenna (1GHz) in reflection, in order to detect the morphology of the section and the details of the masonry leaves and of damaged areas.

### 2.10.8.2.3.3 Thermographic tests

Thermographic tests were performed to achieve information about the masonry texture, hidden by a thick plaster layer. The knowledge of the disposition of bricks or stones and of their shape is important to identify the most representative areas where to place local tests, such as single and double flat jack tests which can give information on the masonry properties.

### 2.10.8.2.3.4 Single and double flat jack tests

Thanks to the results coming from active thermography tests, two areas of the walls having different characteristic were identified. On this two areas other diagnostic tests were carried on: sonic tests to get indications about the morphology of the masonry sections. Then single and double flat Jack test were performed in order to verify the state of stress and its distribution and to study the elastic characteristics of the masonry.

### 2.10.8.2.3.5 Dynamic identification tests

Dynamic identification tests have been performed in the South-East wing of the Spanish fortress. The investigations consisted in the measurement of environmental vibration (natural frequencies and mode shapes) in several points of the structure. Tests were aimed at identifying the dynamic behavior of this part of the building at the present state - i.e. following the severe state of damage caused by the earthquake of April 2009 and subsequent emergency provisional interventions - by means of environmental vibration tests (excitation source: wind, traffic, etc.).
The aim of the test was to evaluate the behavior of the perimeter walls toward inside (courtyard) and outside (ditch) the fortress, after the overturning mechanisms activated by the earthquake. From a structural point of view it was important to know if the two walls (also thanks to the provisional tie-rods system) have different dynamic behavior after the seismic event.

2.10.8.3 Monitoring
In December 2009 a dynamic monitoring system was installed in the Fortress, following a first investigation campaign carried out in September, including dynamic identification tests. The system complements a static monitoring system installed in the first months after the earthquake by the ISCR (Istituto Superiore per la Conservazione ed il Restauro - National Conservation and Restoration Institute) of Rome, devoted to the control of the crack pattern evolution and of the environmental parameters. An acquisition unit connected to eight high sensitivity piezoelectric accelerometers composes the dynamic system. The central unit, located at the second floor of the fortress, in the South-East wing, is provided with a Wi-Fi router for remote data transmission.

A couple of reference sensors is fixed at the base of the structure (at the foot of one of the massive pillars on the inner courtyard, with CH1 orthogonal to the façade and CH2 parallel to it) for the record of the ground acceleration both in operational conditions and during seismic events. The positioning of the acceleration sensors on the elevation of the S-East was decided as a result of the structural dynamic identification. According to the predominant motion direction, sensors were fixed orthogonally to the façade, following vertical and horizontal lines, on the internal and external façades, with an increased number of sensor at the second level.

Dynamic data are being collected both at fixed time intervals ("long" acquisition, corresponding to 131'072 points, or to 21'51'' of record at a sampling frequency of 100 SPS, each 1-24 hours) to allow successive dynamic identification of the structure with different environmental conditions, and on a trigger basis (shorter records, 3'35'' at a sampling frequency of 100 SPS), when the signal, on one of the acceleration channels, gets over the predefined threshold (meaningful event, e.g. earthquake). The latter (triggered) monitoring of the dynamic behavior of the structure requires the definition of a threshold value for the acceleration, which can be settled on every monitored channel and on the time and/or frequency domain.

2.10.8.4 Structural analysis
Local and global FE models are already available. A dead weight analysis has been already performed.

2.10.9 Previous or on-going restoration works

2.10.9.1 Provisional strengthening intervention after the earthquake
In the post-seismic scenario the emergency activities to protect the buildings belonging to cultural heritage have been developed at two parallel levels: (i) damage survey and design, (ii) implementation of temporary safety measures.

The design of temporary interventions for the safety of an historical building starts from the damage survey and from the identification of the collapse mechanisms activated from the seismic action.

According to the observed damage pattern, provisional strengthening interventions were applied to the South-East and South-West wings of the fortress, where the heaviest damages were observed. The strengthening interventions were carried out by different teams of specialized people from the National Fire Brigades (SAF-Speleo-Alpino-Fluviali). The interventions did not rely upon external propping structures, also considered the massive dimensions of the fortress. The structural stability was provided by relying on the remaining strength of the resisting elements, e.g. by connecting the internal and external façades of the damages wings by means of stainless steel cables, in order to avoid the observed overturning mechanisms evolution, especially taking into account that non negligible aftershocks occurred for several months.
In the South-East wing it was necessary to rebuild the roof, by using hollow section steel trusses and a light covering structure made of wood. The substitution of the original wooden structure of the roof with a heavy and stiff reinforced concrete structure, without any strengthening interventions on the underlying masonry walls, caused the collapse of the upper part of the façade. In the South-West wing steel frames were positioned in contrast to the external and internal façades before tensioning the cables (Errore. L'origine riferimento non è stata trovata.).

![Figure 2.228 - Provisional interventions carried out on SE and SW wings: connection of the internal and external façades by means of steel cables and steel frames against the facades.](image)

**2.10.10 Historical research**

It will be necessary to carry out additional historical investigations within the NIKER Project, focusing in particular on the effects of past interventions on the structural response of the structure during the earthquake.

**2.10.11 Local seismicity and characterization of seismic action**

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the fortress is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building's location.

**2.10.12 Proposed (additional) inspection works**

**2.10.12.1 Geometric survey. Aims, methods and technologies**

A complete and rather precise geometric survey of the building is already available. However, a more detailed survey, using also new technologies is considered as an interesting possibility in order to enrich the available geometric information.

**2.10.12.2 Research on materials. Aims, methods and technologies**

Non-destructive technologies resulting for the project will be applied, if possible, for a better characterization of the material physical and mechanical properties.
2.10.13 Proposed monitoring.

2.10.13.1 Static monitoring
The static monitoring already installed in the fortress will be kept active during the project duration. Given its significance, particular attention will be given to the monitoring of the variation of temperature in the building during the project duration. Temperature influences largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the building is necessary for a correct interpretation and post-processing of both results.

2.10.13.2 Dynamic monitoring
The dynamic monitoring already installed in the fortress will be kept active during the project duration.

High density dynamic information will be continuously recorded. This continuous dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for ambient vibration along with its dependence with environmental parameters (temperature, humidity); (2) Capture the dynamic response in the occasion of possible seismic events.

2.10.13.3 Monitoring phases

2.10.13.3.1 Monitoring before intervention
The static and dynamic monitoring systems will be working before the execution of the restoration works and the implementation of the seismic strengthening solutions.

2.10.13.3.2 Monitoring and control during intervention
Both the static and the dynamic system will be active during the execution of the proposed seismic intervention.

2.10.13.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step by step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.10.14 Proposed structural analysis. Aims and methods

2.10.14.1 Modeling and structural analysis methods
It will be necessary to calibrate the available FE models on the results of investigations and monitoring in order to perform advanced linear and non linear FE structural analysis.

Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model) or larger parts of it.

Local analysis will be implemented by means of the kinematic limit analysis method.

2.10.14.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general lay-out of static and dynamic monitoring.
2.10.14.3 Assessment of possible intervention methods
Structural models, using the same modeling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.10.15 Envisaged interventions

2.10.15.1 Considerations on the need for conservation, repair and maintenance
The building has been heavily damaged by the earthquake and urgently needs the design and implementation of strengthening interventions and seismic retrofitting in order to stop the damage evolution.

2.10.15.2 Need for seismic strengthening and proposed actions
The most suitable strengthening interventions will be designed and implemented during the project duration. In particular the design of strengthening intervention will be focused on:
- The pillars of the South-East wing of the fortress.
- Repair and local reconstructions of the damaged masonry structures by means of traditional techniques
- Strengthening of vaults through the application of fibre reinforced composite materials
- Substitution of the heavy RC structures of the roof with a light timber structure.

2.10.16 Potential contribution to the project

2.10.16.1 Compliance with project objectives
The proposed case study gives the opportunity to:
- prove the reliability of the adopted models for the seismic assessment and the structural analysis
- verify the effectiveness of the proposed structural solutions.
- evaluate the application of new technologies and methods developed during the project.

2.10.16.2 Foreseen opportunities for application of new approaches and technologies
This case study can be considered for the application of a knowledge-based methodology developed during the project, which consists in the implementation of a step-by-step procedure or incremental approach. The interventions techniques or the intervention scheme must allow for progressive decision-making and implementation of intervention measures. The final decision is substantially supported by the progressively increasing knowledge of the real response of the building acquired before, during and after any step, by appropriately studying the real case in its real environmental conditions and conditions of use.

Another important aspect concerns the definition of guidelines and criteria for the reconstruction of the damaged parts of the building, indicating the most appropriate strengthening interventions on vertical and horizontal elements according to the techniques tested in the project.

2.10.16.3 Foreseen opportunities for testing and validation of proposed intervention methods
A tentative intervention proposal, aimed at seismic improvement, will be formulated during the project. The intervention proposal will take into account the application of methodologies and criteria resulting from the project. The possibility of alternative solutions, based on innovative technologies will be carefully examined. One of more alternative solutions, based on the project outcome, will be developed into detail for its possible real implementation.

The strengthening intervention may be actually implemented during the project duration (depending on several factors). In this case, the presented case study will also offer an interesting possibility for the evaluation and validation for the proposed intervention method.
2.10.16.4 Potential contribution to validate the project’s methods and criteria

Given the fact that the Spanish fortress has been already investigated by means of some NDT techniques, the focus of further researches should be placed on static and dynamic monitoring technologies oriented to model updating and dynamic characterization.

The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

2.10.16.5 Other specific opportunities provided by the proposed case study

The case study of the Spanish Fortress offers an interesting case regarding the study of the performance, the seismic assessment and the design of strengthening interventions in a building strongly damaged by an earthquake in a post seismic emergency scenario.

It could become a pilot project for the implementation of a standardized procedure in the reconstruction process of the building belonging to cultural heritage in the l'Aquila city centre and in the surrounding historical centers hit by the earthquake.

Table 2.24 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>General geometric survey available</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Stone properties identified based on samples taken from the building. Physical properties of mortar and stone, and mechanical properties of stone will be identified (POLIMI)</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT already performed consisting of sonic pulse velocity tests, radar tests, thermographic tests, single and double flat jack tests, dynamic identification tests</td>
</tr>
<tr>
<td>No additional NDT and MDT are foreseen at the moment.</td>
<td></td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>1 year of continuous static monitoring already available including temperature and humidity sensors, base line extensometers and crackmeters.</td>
</tr>
<tr>
<td>The static monitoring will be kept active during the project duration.</td>
<td></td>
</tr>
<tr>
<td>2.2 Dynamic monitoring</td>
<td>1 year of continuous dynamic monitoring available constituted by eight high sensitivity piezoelectric accelerometers. Several earthquakes captured.</td>
</tr>
<tr>
<td>The dynamic monitoring will be kept active during the project duration.</td>
<td></td>
</tr>
<tr>
<td>3. Structural analysis</td>
<td></td>
</tr>
<tr>
<td>3.1 Modeling and model updating</td>
<td>FE models available</td>
</tr>
<tr>
<td>Need to calibrate the models on the results of investigations and</td>
<td></td>
</tr>
<tr>
<td>Section</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>3.2 Seismic analysis</td>
<td>To be performed using FEM pushover calculations and kinematic limit analysis in combination with Capacity Spectrum Method and the new calculation criteria and methods derived from the Project.</td>
</tr>
<tr>
<td>4. Intervention</td>
<td></td>
</tr>
<tr>
<td>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</td>
<td>Tentative characterization of seismic strengthening needs based on minimal intervention. Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria.</td>
</tr>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td>Design of strengthening interventions regarding the pillars of the South-East wing of the fortress. Repair and local reconstructions of the damaged masonry structures by means of traditional techniques, strengthening of vaults through the application of fibre reinforced composite materials, substitution of the heavy RC structures of the roof with a light timber structure.</td>
</tr>
<tr>
<td>4.2 Real implementation</td>
<td>The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td>Possibility to apply incremental approach based on continuous monitoring. Available for monitoring during and after execution of strengthening.</td>
</tr>
</tbody>
</table>

### References

- AA. VV., Fortezze d’Europa. Forme, professioni e mestieri dell’architettura difensiva in Europa e nel Mediterraneo spagnolo, Atti del convegno (L’Aquila, 6-8 marzo 2002)
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

- Guidelines for evaluation and mitigation of seismic risk to cultural heritage with reference to technical constructions regulation, Ministry for Cultural Heritage and Activities, General Direction of Architectural Heritage and Landscape - Dept. of Civil Protection Agency (Italy)
- Modello B-DP PCM-DPC MiBAC 2006 - Scheda per il rilievo del danno ai beni culturali - Palazzi. Source: www.protezionecivile.it
2.11 THE CIVIC TOWER - L’AQUILA - ITALY

2.11.1 Name, location and short description

The Civic Tower of L’Aquila was built before the extension of the city of L’Aquila in 1254. The slender stem was erected between 1254 and 1374 with a beautiful arrangements of square blocks of stone. In 1374 the captain Tommaso degli Albizi installed on the top of the tower one of the first invented clock, immediately after the ones of Florence and Ferrara.

When the tower was erected in 1254 it was 70 meters high, but after the earthquake of 1703 it was reconstructed much smaller. Its height was further reduced in 1838 due to structural damages and its final part was substituted by a small terrace.

The last documented restoration date back to 1937 when some strengthening intervention were performed.

During the 6th of April 2009 earthquake the tower was heavily damaged and some provisional interventions was necessary in order to avoid further collapses.

2.11.1.1 Geometrical, structural and material features

A preliminary geometric survey is available.
Figure 2.229 - Palazzo Margherita and the Civic Tower: plan view of the ground floor.

Figure 2.230 - Palazzo Margherita and the Civic Tower: North elevation.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

Figure 2.231 - Palazzo Margherita and the Civic Tower: East elevation.

Figure 2.232 - Present condition of the tower after the earthquake.
2.11.2 Present and foreseen future use. People at risk.

The Civic tower was severely damaged by the earthquake in 2009 and classified unfit for use since then. Nobody is allowed to enter the building due to the critical structural conditions of the internal structures.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterized by different coefficients of use (Table 2.27).

Table 2.25 Classes and coefficients of use $C_U$ for buildings.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use $C_U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The civic tower is classified in the 3rd class of use, with a coefficient of use $C_U = 1.5$.

The nominal life, according to the type of construction, is defined as $V_N \geq 50$ years. The reference period for the seismic action is given by the product $C_U \times V_N = 75$ years.

2.11.3 Considerations on valuable cultural content

The study on the valuable cultural content is to be performed within the project.

The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:

- Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
- Categories of use (occasional or unused, frequent and very frequent)

The combination of these categories leads to define the exceeding probability (P) of the seismic action in 50 years and the importance factors $\gamma_i$ for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.11.4 Information on local seismicity

2.11.4.1 Local seismicity

The city of l’Aquila is located in the central part of Italy, in the Abruzzo region. It is a high seismic zone characterized by a PGA (Peak Ground Acceleration) of 0.261g for a 475-year return period according to the Italian Code NTC 2008.

This region of central Apennines has one of the highest seismic hazard in Italy. Many destructive earthquakes are filed in the historical catalogues. The last dramatic event was the MW 6.7, 1915 Avezzano earthquake, located about 30 km to the south-west, that caused about thirty thousand
Large earthquakes originate mainly on a narrow belt along the central Apennines, accommodating a NE-trending extension, with a rate of about 3 mm/yr, progressively thinning the Apennines thrust and fold belt.

2.11.4.2 Characterization of the seismic action during the 6th of April 2009 earthquake

On the 6th April 2009, a Mw 6.3 (MI 5.8) earthquake caused significant damage to L’Aquila, the medieval capital city of the Abruzzo region of Italy, and several surrounding towns and villages. 297 people were killed, 1,000 injured, 66,000 made homeless, and many thousands of buildings were destroyed or damaged.

With this seismic event the toll in terms of structural damage was enormous. In general, all the masonry buildings suffered a great amount of damage, in great part due to the fact that a vast amount of this buildings were made of poorly arranged masonry composed by round pebbles and mortar of scarce mechanical characteristics.

The L’Aquila earthquake presented a Richter Magnitude (MI) of 5.8, while strong-motion data inversion resulted in a moment magnitude (Mw) of 6.3 with a shallow focal depth, of approximately 8.0 to 9.0km. The epicentre was located 10.0 km West of L’Aquila and 95.0 km NE of Rome. The main shock seemed to have been originated on a direct fault (the Paganica fault), 15.0 km long in NW-SE direction.

The earthquake involved a wide area among the cities of L’Aquila, Avezzano, Sulmona and Teramo. The ground morphology played an important role in the structural damage distribution and the most catastrophic effects were observed along the Aterno river valley.

2.11.5 Previous and on-going studies

2.11.5.1 Inspection works

2.11.5.1.1 Geometric survey

A rather precise geometric survey of the entire building is available.

2.11.6 Previous or on-going restoration works

2.11.6.1 Provisional strengthening intervention after the earthquake

Some provisional strengthening interventions have been already performed involving both the ‘Margherita’ Palace and the Civic Tower. During the project a detailed description of these interventions will be given.

2.11.7 Historical research

It will be necessary to carry out exhaustive historical investigations within the NIKER Project, focusing in particular on the effects of past interventions on the structural response of the structure during the earthquake.

2.11.8 Local seismicity and characterization of seismic action

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the building is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building’s location.
2.11.9 Proposed (additional) inspection works

2.11.9.1 Geometric survey. Aims, methods and technologies
It is necessary to perform a complete and precise geometric survey of the building, using also new technologies developed during the project in order to enrich the available geometric information.

2.11.9.2 Deep inspection by means of NDT and MDT. Aims, methods and technologies
Within the NIKER project it will be possible to perform an extensive investigations’ campaign in order to characterize both quantitatively and qualitatively the masonry structures of the building and its global and local structural behavior.

Deep inspections including both NDT and MDT are foreseen, such as: sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests,.. and other possible new investigation techniques derived from the project.

In particular the main objective is to concentrate the research activities on the dynamic survey of the building using ambient vibration tests. These tests are necessary to design and install the proposed monitoring systems.

2.11.10 Proposed monitoring

2.11.10.1 Static monitoring
A static monitoring will be installed in the tower during the project including devices to measure a set of displacements and strains at critical points of the buildings.

The static monitoring system is composed by 18 channels:

- 5 strain gauges on the existing metal ties of the tower to control the strain variation.
- 1 inclinometer to control the displacement of the tower in the two in-plane directions.
- 3 displacements transducers installed on representative cracks in the lower part of the tower to control the crack width.
- 2 displacements transducers installed at the gap between tower and palace to control the relative displacements of the two structures
- 6 thermal sensors to control both the air temperature and the walls temperature at different points of the structure

Data from the static system will be registered every 30 minutes.

Given its significance, particular attention will be given to the monitoring of the variation of temperature in the building during the project duration. Temperature influences largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the building is necessary for a correct interpretation and post-processing of both results.

The static monitoring will be kept active during the entire duration of the project.

2.11.10.2 Dynamic monitoring
The proposed dynamic monitoring is composed by 8 high sensitivity piezoelectric accelerometers connected to an acquisition unit with a Wi-Fi router for remote data transmission. Three reference sensors are fixed at the base of the structure to record the ground acceleration both in operational conditions and during seismic events. The positioning of the other acceleration sensors on tower’s elevation will be decided on the results of the structural dynamic identification.

High-density (100 sps) dynamic information will be continuously recorded. This continuous dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for ambient vibration along with its dependence with environmental parameters (temperature, humidity); (2) Capture the dynamic response in the occasion of possible seismic events.

2.11.10.3 Monitoring phases
2.11.10.3.1 Monitoring before intervention
The static and dynamic monitoring systems will be working before the execution of the strengthening interventions and restoration works that are going to be performed soon.

2.11.10.3.2 Monitoring and control during intervention
Both the static and the dynamic system will be kept active during the execution of the proposed interventions.

2.11.10.3.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step-by-step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.11 Proposed structural analysis. Aims and methods

2.11.11 Modeling and structural analysis methods
It will be necessary to create and implement FE models of the tower and to calibrate them on the results of investigations and monitoring in order to perform advanced linear and non linear FE structural analysis.

Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model). Local analysis will be implemented by means of the kinematic limit analysis method.

2.11.11.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general layout of static and dynamic monitoring.

2.11.11.3 Assessment of possible intervention methods
Structural models, using the same modeling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.11.12 Envisaged interventions

2.11.12.1 Considerations on the need for conservation, repair and maintenance
The building has been heavily damaged by the earthquake and urgently needs the design and implementation of strengthening interventions and seismic retrofitting in order to stop the damage evolution.

2.11.12.2 Need for seismic strengthening and proposed actions
The most suitable strengthening interventions involving the entire restoration and/or reconstruction of the collapsed parts of the tower will be designed and implemented during the project.

2.11.13 Potential contribution to the project

2.11.13.1 Compliance with project objectives
The proposed case study gives the opportunity to:
prove the reliability of the adopted models for the seismic assessment and the structural analysis
verify the effectiveness of the proposed structural solutions.
evaluate the application of new technologies and methods developed during the project.

2.11.13.2  Foreseen opportunities for application of new approaches and technologies
The building can be considered for the application of new non-destructive technologies for
material, mechanical and morphological characterization.
Significant effort will be carried out on the characterization of the structural response by means of
both static and dynamic monitoring, and the use of the monitoring output for model updating and
validation. Technologies for dynamic monitoring (as described in section 6.2) and for structural
analysis will be tested and evaluated. Particular attention will be given to the combined use of
monitoring and structural analysis through an integrated methodology allowing sound validation of
structural models.

2.11.13.3  Foreseen opportunities for testing and validation of proposed intervention
methods
A tentative intervention proposal, aimed at the restoration, reconstruction and seismic
improvement, will be formulated during the project. The intervention proposal will take into account
the application of methodologies and criteria resulting from the project. The possibility of alternative
solutions, based on innovative technologies will be carefully examined. One of more alternative
solutions, based on the project outcome, will be developed into detail for its possible real
implementation.
The strengthening intervention may be actually implemented during the project duration
(depending on several factors). In this case, the presented case study will also offer an interesting
possibility for the evaluation and validation for the proposed intervention method.

2.11.13.4  Potential contribution to validate the project's methods and criteria
The focus of further researches should be placed on the implementation of NDT and MDT tests for
the characterization of the material and structure properties and on static and dynamic monitoring
technologies oriented to model updating and dynamic characterization.
The building can be considered as well for the selection, design, and application of minimized
interventions and for the evaluation intervention strategies, including possible step-by-step
approaches. It is expected that the presented case study will contribute very significantly to the
calibration and validation of the general methodology, tools and criteria resulting from the project
thanks to the opportunities it offers for their real implementation.

2.11.13.5  Other specific opportunities provided by the proposed case study
The case study of the Civic Tower offers an interesting case regarding the study of the
performance, the seismic assessment and the design of strengthening interventions in a building
strongly damaged by an earthquake in a post seismic emergency scenario.
It could become a pilot project for the implementation of a standardized procedure in the
reconstruction process of the building belonging to cultural heritage in the l'Aquila city centre and in
the surrounding historical centers hit by the earthquake.

2.11.14 Needs for cooperation from other partners

2.11.14.1  Capabilities of the local group regarding the application of methods and
technologies
UNIPD counts on the personnel, experience and the equipment necessary for the following
proposed activities:
  • Sonic tests, flat jack tests and dynamic identification tests equipment.
• Dynamic monitoring by means of high sensitivity piezoelectric accelerometers
• Static monitoring including temperature and humidity sensors, strain gauges, displacement transducers and crackmeters, inclinometers.
• Structural analysis by means of non-linear FEA and model updating

Table 2.26 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>Complete and precise geometric survey of the building</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Not expected within the project</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT consisting of sonic pulse velocity tests, single and double flat jack tests, dynamic identification tests. The research effort will concentrate in particular on the dynamic survey of the building.</td>
</tr>
</tbody>
</table>

2. Monitoring

| 2.1 Static monitoring | A continuous static monitoring system is currently being installed including temperature and humidity sensors, linear displacement transducers on the main cracks, strain gauges on steel ties and one tiltmeter. |
| 2.2. Dynamic monitoring | A continuous dynamic monitoring system is currently being installed including eight high sensitivity piezoelectric accelerometers. |

3. Structural analysis

| 3.1 Modeling and model updating | FE models creation and implementation. Need to calibrate the models on the results of investigations and monitoring. Model updating techniques exploiting the monitoring results. |
| 3.2 Seismic analysis | To be performed using FEM pushover calculations and kinematic limit analysis in combination with Capacity Spectrum Method and the new calculation criteria and methods derived from the project. |

4. Intervention

| 4.1 Characterization of needs for repair, maintenance and (seismic) strengthening | Tentative characterization of seismic strengthening needs based on minimal intervention. Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria. |
4.1 Design of strengthening intervention | Design of strengthening interventions and seismic retrofitting to be performed during the project.

4.2 Real implementation | The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.

4.3 Monitoring during and after strengthening implementation | Possibility to apply incremental approach based on continuous monitoring. Available for monitoring during and after execution of strengthening.

2.11.15 References

- Guidelines for evaluation and mitigation of seismic risk to cultural heritage with reference to technical constructions regulation, Ministry for Cultural Heritage and Activities, General Direction of Architectural Heritage and Landscape - Dept. of Civil Protection Agency (Italy)
- Modello A-DC / B-DP PCM-DPC MiBAC 2006 - Scheda per il rilievo del danno ai beni culturali - Chiese e Palazzi. source: www.protezionecivile.it
2.12 THE CHURCH OF S.BIAGIO AMITERNUM AND THE ORATORY OF S.GIUSEPPE DEI MINIMI, L’AQUILA, ITALY

2.12.1 Name, location and description

2.12.1.1 The church of S.Biagio Amiternum
San Biagio Church is located in the historical city centre of L’Aquila in the quarter of San Pietro close to the apse of the Santi Giorgio e Massimo Cathedral. The structure is contiguous to the San Giuseppe dei Minimi Oratory by which it shares a wall. The façade is approximately north-facing and gives onto Via Sassa, an historical street connecting Piazza Duomo to Porta Barete.

The Church is part of an aggregate even if it has the same depth of the block where it is inserted. The back façade faces on via del Cembalo di Colantoni; eastward the Church borders on the Oratory and other ordinary buildings, the latter present on the west side too.

San Biagio Church has a great importance in the history of L’Aquila since it is associated to the foundation of the city as fusion of pre-existing castles and two dioceses. The building underwent two relevant interventions in the last 30 years and since 9/04/2010 it has been stated unsafe further to the seismic event. The first intervention was the protection of the collapsed upper part of the façade from the rain water.

2.12.1.2 The oratory of S.Giuseppe dei Minimi
The Oratory of S. Giuseppe dei Minimi is located in the heart of the city of L'Aquila. The building is adjacent to the church of San Biagio d'Amiternum, with which he shared a wall.

The Oratory is of considerable importance in L'Aquila and the original framework dated back to the XIII century. From an architectural point of view is one of the most important baroque buildings. From a cultural point of view is the seat of the musical company “Solisti Aquilani.”

The southern façade of the oratory overlooks Via Roio, while the eastern longitudinal wall overlooks Via degli Scardasseri. The apse borders ordinary buildings toward north and the western walls is in common with the church of S.Biagio.

The building is located on the corner of an aggregate of historical buildings and about ten years ago it was restored. The Oratory, after the earthquake of 6 April 2009, was judged unfit for use and some provisional strengthening interventions were performed on its facade.

2.12.2 Historical note
2.12.2.1 The church of S.Biagio Amiternum

From the early days of the conurbation Aquila had an important church dedicated to San Vittorino, after referred as, no one knows why, San Biagio di Amiterno.

The church of San Biagio di Amiterno (formerly San Vittorino) is located in the oldest part of the city of L’Aquila, in the XIII century where the urban structure dating from the XIII-XIV century lasted like the road network, almost to this day.

The ecclesiastical complex stands on the sidelines of the quarter of San Pietro di Coppito and is part of a block ranging from via Cembalo Colantoni, via Roio, apse on Via Sassa and Piazza San Biagio, which is located on the main façade.

This position places it in close relation with the adjacent oratorio dedicated to San Giuseppe dei Minimi and the nearby church dedicated SS. Giorgio e Massimo, Duomo of the city of L’Aquila.

The reconstruction of the city by the Angioini, carried out from 1267, included the rebuilding of the ancient church of San Vittorino, probably further downstream of the building.

The church has gone through the centuries suffering considerable damage from earthquakes. The first reconstruction was in 1267 after the destruction by Manfredi in 1259, and in 1315 had to endure the first great earthquake, which destroyed it almost completely. Back on the reconstruction by the residents of Amiterno started operating in 1326, but it was not long time that another disastrous earthquake in 1349 brought new damage to the church. It is clear that the church in all the general plants of the historical city of Aquila, until the end of 1600, has a facade that faces west, and only later in the church is represented with a plan rotated from east-west to north-south.

It is not certain whether such rotation has taken place before the fourteenth century or after 1461, when a new earthquake destroyed L’Aquila and San Biagio, but unfortunately the documentary sources are not helpful. In fact, the information referring to the 1400 and 1500 are very little if we exclude those relating to the decorations carried out on the walls which no longer exist, and attributed at the Alferi the artists who painted the frescoes in Santa Giusta.
Between 1629 and 1661 the building underwent a complete renovation with the internal space very close to what is described by the term "new" Baroque, a series of chapels were to occupy the side aisles and the whole building was redesigned, as appears from the documents found by the historian Colapietra, by Francesco Bedeschini from Aquila distinguished artist of the time.

The earthquake of 1703 left irreversible damage to any part of the building both the fourteenth-century structures and those of the late seventeenth century. The rebuilt was rather long in time and probably the structure and the space ultimately proved to be different from what was originally designed and developed.

The decline of the church of San Biagio began just after the reconstruction of 1754 was completed. It lost all its privileges, which lasted for about 500 years, with main the lost of the parish of Amiternini and in a few years its operation ended. The church was abandoned during the First World War becoming soldiers' dormitory, and in recent years of the last century the exhibitions and markets that took place resulted in a continual state of decay and neglect.

2.12.2 The oratory of S.Giuseppe dei Minimi

The first construction of the oratory dates back to 1646, when a part of the previous St. Biagio church was given to the brotherhood of the Suffragio, which decided to build a new, small church, however maintaining some features of the ancient one, such as the two lancet windows and the thirteenth century portal in the facade on via Roio. The church was opened in 1649, but construction works continued up to 1701. After only 2 years, the earthquake of 1703 induced severe damage on the church. Finally, in 1770 the building was sold to the brotherhood of S. Giuseppe dei Minimi.

2.12.3 Geometrical, structural and material features

2.12.3.1 The church of S.Biagio Amiternum

San Biagio is a church with one nave and two aisles, each one ending with an apse and crossed by a transept not much protruding. The building is 28 m. long, 21 m. wide and 17,5 m high from the roof to the upper part of the roof.

The aisles are covered by little brick dome as well the intersection of the transept with the little navies. The lateral apses are covered with barrel vaults; in addition the right one presents stone ribs. The central navy has barrel vault with lunette made of reeds and gypsum. This vault becomes a lower dome at the joint between the dome and the transept and continues over the presbytery until the apse covered with an eight side hip roof.
The outer part of the façade of the Church is 8 degrees rotated compared to the inner one in order to cover the difference between the axis of the building and the street. The configuration of the façade reflects the section of the Church; the upper part is rustic while the remaining part is covered with plaster. From the right part of the transept it is possible to enter a sacristy covered by a hip vault. The room has a square plan whose side is approximately 6.3 m. In the middle between Via del Cembalo di Colantoni and the Sacristy there are other three secondary rooms.

Figure 2.235 - Plan view: the church of S.Biagio (on the right) and the oratory of S.Giuseppe (on the left).

Figure 2.236 - Transversal section of the church.
Figure 2.237 - North elevation: the main facade of the church.

Figure 2.238 - Left aisle of the church (a). Barrel vault of the main nave (b).
2.12.3.1.1 Masonry walls

The walls of the main façade are three layered walls with the inner layer made by irregular elements and a large amount of mortar (rubble inside layer), probably with a low mechanical capacity and poor connection between leaves (Errore. L'origine riferimento non è stata trovata.).

The other lateral façade along Cembalo di Colantoni Street is a masonry wall made with small sized irregular stones as we can see in Errore. L'origine riferimento non è stata trovata.. This particular masonry was common in the 15th and the 17th century in L’Aquila, and it can be noticed also on external face of the longitudinal walls, above the lateral naves.

2.12.3.1.2 Vaults

The central nave is surmounted by a ‘camorcanna’ vault, made by wooden ribs, canes mat and plaster (Figure 2.241). The canes are longitudinal spitted, so even if it is not possible to directly measure the diameter of it, it must be considerable (maybe 20-25 mm).

The lateral naves are covered by hemispherical cupolas, which become elliptical and are rising while intersecting the transept. These cupolas appear to be real, built in brick masonry and in some areas the detachment of plaster and some damages are visible.
2.12.3.1.3 Roof

The San Biagio church roof was made of wooden trusses with purlins (pane), rafters and tiles. Above the original roof of the church, in 1980 a new roof was built with prestressed rafters and hollow bricks, with a bead perimeter and metal tie-rods.

2.12.3.2 The oratory of S. Giuseppe dei Minimi

The plan of the Oratory of S. Giuseppe has a single room, with half circles at the end of the short sides (Errore. L'origine riferimento non è stata trovata.). A barrel vault covers the main hall of the oratory (Figure 2.243). The length of the building is approximately 21 m, it is 14 m wide, and the height from the floor to the roof ridge is approximately 14.5 m.
2.12.3.2.1 Masonry walls
The walls of the oratory are composed by stone masonry with irregular and small ashlars. This kind of masonry is particularly widespread in the sixteenth and eighteenth centuries in L'Aquila. It also appears on the outer face of the wall above the adjacent longitudinal aisle of St. Biagio.

2.12.3.2.2 Vaults and roof
The structure of the roof of the Oratory is unknown. Due to the presence of a flat ceiling, before the restoration works at the end of the 90's, it is possible to assume the presence of wooden trusses. It was not possible to establish the structural characteristics of the main vault; however, based on historical information and damage survey, it should be a false ceiling.

2.12.4 Present and foreseen use. People at risk.
The church and the oratory have been used, since their construction, for the corresponding liturgical purposes. They were severely damaged by the earthquake in 2009 and classified unfit for use since then. Nobody is allowed to enter the church apart from the workers involved in the restoration works.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterized by different coefficients of use (Table 2.27).

Table 2.27 - Classes and coefficients of use $C_U$ for buildings.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use $C_U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The church and the oratory are classified in the 3rd class of use, with a coefficient of use $C_U = 1.5$. Their nominal life, according to the type of construction, is defined as $V_N \geq 50$ years. The reference period for the seismic action is given by the product $C_u \times V_N = 75$ years.

2.12.5 Considerations on valuable cultural contents

The study on the cultural contents will be performed during the project development. The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:
- Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
- Categories of use (occasional or unused, frequent and very frequent)

The combination of these categories leads to define the exceeding probability ($P$) of the seismic action in 50 years and the importance factors $\gamma_I$ for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.12.6 Present condition and damage

2.12.6.1 Considerations on the present condition

2.12.6.1.1 The church of S.Biagio Amiternum

During the first survey, the damage to the church and the oratory was classified as “very serious”, rating 4 in a scale of 5, where the maximum grade would correspond to extended or global collapse. Although in San Biagio some elements had collapsed, fortunately most of the widespread cracking had not evolved into a complete mechanism. The building was initially declared not accessible. Provisional interventions amounted to removing or fixing some tottering parts and closing the large opening caused by the collapse of part of the façade, in order to prevent environmental damage. Afterwards, controlled access to the interior was permitted only for technical purposes and study.
2.12.6.1.2 The oratory of S. Giuseppe dei Minimi

The oratory of St. Giuseppe dei Minimi in L'Aquila, Italy, reported serious damage, as many other churches of the city, after the seismic events of the April 2009, although the overall performance of the church was quite satisfactory as it survived the earthquake (magnitude 5.8 on the Richter scale).

2.12.6.2 Main observed damage and decay

2.12.6.2.1 The church of S. Biagio Amiternum

The main damage occurred after the main shock and consisted in the out-of-plane collapse of the upper part of the façade and of the corresponding infill in the church interior, Errore. L'origine riferimento non è stata trovata.. Debris has cumulated outside and inside. Failure occurred according to a well-known mechanism, also represented in the figure.

Figure 2.245 - The top of the façade (left) and corresponding collapse mechanism (right).

In the church interior, local collapse occurred at different spots of the barrel vault over the main nave, bringing into evidence its cane structure. This damage was likely due to debris falling from the roof area and breaking through.

Figure 2.246 - The damages in the church.

Again in the interior, widely extended cracking had occurred (Figure 2.246). All the visible crack systems have been examined and rendered. In particular, cracking occurred - in the lateral walls of the central nave, above the arches, where cracks developed systematically from the arch key area to the windows above with a moderate slant with respect to the vertical direction. In the lateral naves, where all the spherical vaults over the chapels cracked along a parallel just above the base; additionally, the arches between chapels were cracked at the key, in a system connecting with the vault cracks.
In the exterior walls of the lateral naves, where widespread cracks appeared, these cracks seem, however, narrower and shallower.

At the base of the pillars, with generally thin vertical cracks. Damage was mainly found in the layer of stiff decorative plastering, which at places detached into bubbles and broke off in others. No systematic inspection of the inner structure was possible.

2.12.6.2.2 The oratory of S. Giuseppe dei Minimi

The façade of the oratory of St. Giuseppe dei Minimi was subjected, due to the seismic forces, to an overturning mechanism towards the outside, as indicated by the wide cracks on the two side walls, close to the corners (Figure 2.247). Quite likely, the presence in the S-W corner of the overhanging belfry structure worsened the situation, since a massive pillar of the belfry acted as a wedge for the crack below. Few other damage mechanisms were detected, such as shear damage in the façade and in the back wall of the apse.

![Figure 2.247 - Wide cracks on both sides of the façade, indicating the detachment of the latter from the rest of the building.](image)

![Figure 2.248 - Damages on the belfry.](image)

2.12.7 Information on local seismicity

2.12.7.1 Local seismicity

The city of l'Aquila is located in the central part of Italy, in the Abruzzo region. It is a high seismic zone characterized by a PGA (Peak Ground Acceleration) of 0.261g for a 475-year return period according to the Italian Code NTC 2008 (Norme Tecniche per le Costruzioni DM 14-01-2008).
This region of central Apennines has one of the highest levels of seismic hazard in Italy. Many destructive earthquakes are filed in the historical catalogues. The last dramatic event was the MW 6.7, 1915 Avezzano earthquake, located about 30 km to the south-west that caused about thirty thousand deaths. Large earthquakes originate mainly on a narrow belt along the central Apennines, accommodating a NE-trending extension, with a rate of about 3 mm/yr, progressively thinning the Apennines thrust and fold belt.

2.12.7.2 Characterization of the seismic action during the 6th of April 2009 earthquake

On the 6th April 2009, an Mw 6.3 (MI 5.8) earthquake caused significant damage to L'Aquila, the medieval capital city of the Abruzzo region of Italy, and several surrounding towns and villages. 297 people were killed, 1,000 injured, 66,000 made homeless, and many thousands of buildings were destroyed or damaged. With this seismic event the toll in terms of structural damage was enormous. In general, all the masonry buildings suffered a great amount of damage, in great part due to the fact that a vast amount of this buildings were made of poorly arranged masonry composed by round pebbles and mortar of scarce mechanical characteristics.

The L'Aquila earthquake presented a Richter Magnitude (MI) of 5.8, while strong-motion data inversion resulted in a moment magnitude (Mw) of 6.3 with a shallow focal depth, of approximately 8.0 to 9.0km. The epicentre was located 10.0 km West of L'Aquila and 95.0 km NE of Rome. The main shock seemed to have been originated on a direct fault (the Paganica fault), 15.0 km long in NW-SE direction.

The earthquake involved a wide area among the cities of L'Aquila, Avezzano, Sulmona and Teramo. The ground morphology played an important role in the structural damage distribution and the most catastrophic effects were observed along the Aterno river valley.

2.12.8 Previous and on-going studies

2.12.8.1 Introduction

Detailed studies involving inspections have been carried out after the earthquake starting from May 2010. In order to get a good characterization of the wall sections and determining the solicitations acting on the structure, NDT and MDT techniques have been used to deeply know the actual state of pillars and walls.

The investigation campaign has been carried out during May 2010 according to the following procedure. Firstly non-destructive tests NDT (sonic test, thermography, radar test) were performed to acquire information both on the elements constituting the masonry and the internal morphology. Then in few selected areas endoscopies, coring and videoendoscopies were made to confirm the results of the NDT. Lastly in some particularly relevant points minor destructive tests MDT (single/double flat jack test) have been performed to obtain information on the mechanical properties of the masonry. In the same points a dismantling of the masonry has been accomplished to verify the morphology of the section.

This procedure has been followed both for the church of S. Biagio and for the adjacent oratory of San Giuseppe dei Minimi.

2.12.8.2 Inspection works

2.12.8.2.1 Geometric survey

A detailed geometric survey of the entire building is available. Additionally, detailed structural and damage survey have been carried out.

2.12.8.2.2 Visual inspection

Detailed visual inspection has been carried out on the entire building (involving all walls, buttresses, vaults, arches and piers) for the purpose of identifying morphology, materials and damage. Cracking and damage has been mapped in detail.
2.12.8.2.3 Deep inspection by means of NDT and MDT

A series of NDT and MDT were performed by the research groups of the Politecnico of Milan and the University of Padova in order to characterize and evaluate quantitatively and qualitatively the state of damage of the masonry structures and to identify the structural response of the most damaged parts of the two buildings.

- The experimental campaigns included:
  - Sonic pulse velocity tests
  - Radar tests
  - Thermographic tests
  - Endoscopies
  - Core drilling
  - Inspections of the masonry section and sampling
  - Single and double flat jack tests

2.12.8.2.4 Dynamic identification tests

2.12.8.2.4.1 The church of S.Biagio Amiternum

Results emerged from the tests applied on the structures of the St. Giuseppe dei Minimi oratory proposed useful information on the constituting masonry typology, analyzed both via non-destructive and minor destructive techniques. The layout of the applied techniques is reported in Figure 2.249.

Figure 2.249 - Plan of the church of S. Biagio: tests layout.
2.12.8.2.4.2 The oratory of S. Giuseppe dei Minimi

Results emerged from the tests applied on the structures of the St. Giuseppe dei Minimi oratory proposed useful information on the constituting masonry typology, analyzed both via non-destructive and minor destructive techniques. The layout of the applied techniques is reported in Figure 2.250.

Figure 2.250 - Plan of the oratory of S. Giuseppe: tests layout.

2.12.8.3 Structural analysis

Global FE models are already available. A dead weight analysis has been already performed.

2.12.9 Previous or on-going restoration works

2.12.9.1 Provisional strengthening intervention after the earthquake

In the post-seismic scenario the emergency activities to protect the buildings belonging to cultural heritage have been developed at two parallel levels: (i) damage survey and design, (ii) implementation of temporary safety measures.

The design of temporary interventions for the safety of an historical building starts from the damage survey and from the identification of the collapse mechanisms activated from the seismic action.

The provisional strengthening interventions included:

- Creation of a provisional covering on the collapsed part of the façade of the S. Biagio church;
- Insertion of metal ties and polyester bands to contrast the overturning mechanism of the façade of the S. Giuseppe oratory;
- Creation of a wooden structure to support the belfry of the oratory.
2.12.10 Historical research

It will be necessary to carry out additional historical investigations within the NIKER Project, focusing in particular on the effects of past interventions on the structural response of the structure during the earthquake.

2.12.11 Local seismicity and characterization of seismic action

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the church is located is not disregarded in order to determine demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building's location.

2.12.12 Proposed (additional) inspection works

2.12.12.1 Geometric survey. Aims, methods and technologies

A complete and rather precise geometric survey of the building is already available. However, a more detailed survey, using also new technologies is considered as an interesting possibility in order to enrich the available geometric information.

2.12.12.2 Research on materials. Aims, methods and technologies

Laboratory tests will be performed on the samples of mortar and stone taken from different points of the structures. Non-destructive technologies resulting for the project will be applied, if possible, for a better characterization of the material physical and mechanical properties.

2.12.13 Proposed monitoring.

2.12.13.1 Static monitoring

A static monitoring (Figure 2.252) will be installed in the church and in the oratory during the project including devices to measure a set of displacements and strains at critical points of the buildings.

The static monitoring system is composed by 8 displacements transducers installed on representative cracks of the two buildings and 2 temperature and humidity sensors to control the environmental parameters.
Given its significance, particular attention will be given to the monitoring of the variation of temperature in the building during the project duration. Temperature influences largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the building is necessary for a correct interpretation and post-processing of both results.

The static monitoring will be kept active during the entire duration of the project.

### 2.12.13.2 Dynamic monitoring

The proposed dynamic monitoring (Figure 2.252) is composed by 6 high sensitivity piezoelectric accelerometers connected to an acquisition unit with a Wi-Fi router for remote data transmission. Two reference sensors are fixed at the base of the structure to record the ground acceleration both in operational conditions and during seismic events. Two couple of accelerometers will be placed at the top of the façade of S. Biagio and S. Giuseppe respectively.

High-density (100 sps) dynamic information will be continuously recorded. This continuous dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for ambient vibration along with its dependence with environmental parameters (temperature, humidity); (2) Capture the dynamic response in the occasion of possible seismic events.

#### 2.12.13.2.1 Monitoring by instrumented anchorage

A monitoring anchorage, jointly developed by UBATH and CINTEC within the framework of WP6.2, will also be installed in the oratory of S. Giuseppe dei Minimi in correspondence of the vertical crack between the main façade and east side wall. The anchoring system includes a dissipative anchor device, previously developed by UBATH and CINTEC (Paganoni et al., 2010), and various sensors, as described in the relevant Inventory Technology Form. In particular, the anchorage includes a triaxial accelerogram, temperature and humidity gauges and a set of strain gauges placed at various locations in the anchorage assembly to the purpose of recording deformation occurring in the anchorage and thus relative movements between the two masonry panels.

![Figure 2.252 - Layout of the static and dynamic monitoring system to be installed in the church of S. Biagio and in the oratory of S. Giuseppe.](image-url)
2.12.13.3 Monitoring phases

2.12.13.3.1 Monitoring before intervention
The static and dynamic monitoring systems as well as the instrumented anchorage will be working before the execution of the restoration works and the implementation of the seismic strengthening solutions.

2.12.13.3.2 Monitoring and control during intervention
The static and the dynamic systems as well as the instrumented anchorage will be active during the execution of the proposed seismic intervention. Depending on the nature of the intervention works to be carried out on the façade of the oratory, the instrumented anchorage may need partially dismantling and reassembling, thus causing an interruption of the data recording process. However, appropriate measurements will be taken before and after the intervention to ensure that the anchorage system is calibrated taking into account the modifications occurred during the repair intervention. Further help in this sense will come from the continuous data records from the dynamic and static monitoring systems.

2.12.13.3.3 Monitoring and survey after intervention
It is intended to have the static and dynamic monitoring systems as well as the instrumented anchorage active during a period of, at least, two years after the implementation of the seismic strengthening. In case of envisaging and applying a possible step-by-step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.12.14 Proposed structural analysis. Aims and methods

2.12.14.1 Modelling and structural analysis methods
It will be necessary to calibrate the available FE models on the results of investigations and monitoring in order to perform advanced linear and non-linear FE structural analysis. Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. First of all it is intended to perform natural frequency analyses on the available model for the calibrations on the results of dynamic identification tests. Then non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model) or larger parts of it. Partial FE models involving the anchorage and the connection between the façade and the side wall will be developed accordingly to the general model of the whole building and calibrated on the basis of the data recorded by both static and dynamic systems as well as by the sensors embedded in the anchorage.

Local analysis will be implemented by means of the kinematic limit analysis method.

2.12.14.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general layout of static and dynamic monitoring. The comparison between the data recorded by sensors embedded in the instrumented anchor and data recorded by static and dynamic systems will help to calibrate the instrumented anchorage and derive the rules for the correlation between the global response of the structure and the response at the connection between façade and side wall as recorded by the anchorage. This will also support the validation process of the instrumented anchorage and amend the design in order to obtain a full characterisation of the behaviour of the structural connection. The output of the calibration process will feed into WP6.2.
2.12.14.3 Assessment of possible intervention methods

Structural models, using the same modelling approaches and tools described in section 12.1, will be also utilized in order to simulate and pre-evaluate the proposed strengthening techniques.

2.12.15 Envisaged interventions

2.12.15.1 Considerations on the need for conservation, repair and maintenance

The building has been heavily damaged by the earthquake and urgently needs the design and implementation of strengthening interventions and seismic retrofitting in order to stop the damage evolution.

2.12.15.2 Need for seismic strengthening and proposed actions

Strengthening interventions will be designed and implemented during the project duration.

2.12.15.2.1 The church of S.Biagio Amiternum

For the church of S. Biagio the design of strengthening intervention will be focused on:

- The substitution of the RC roof with a light timber structure
- Strengthening of vaults of the main nave and of the lateral aisles through the application of fibre reinforced composite materials.
- Strengthening of the pillars
- Insertion of steel ties at different levels
- Repair and reconstructions of the masonry façade by means of traditional techniques

2.12.15.2.2 The oratory of S. Giuseppe dei Minimi

For the oratory of S. Giuseppe the design of strengthening intervention will be focused on:

- Strengthening of the existing timber roof
- Substitution of the existing RC tie beam with a reinforced masonry tie beam
- Insertion of steel ties at different levels
- Intervention on the bell fry

2.12.16 Potential contribution to the project

2.12.16.1 Compliance with project objectives

The proposed case study gives the opportunity to:

- prove the reliability of the adopted models for the seismic assessment and the structural analysis
- verify the effectiveness of the proposed structural solutions.
- evaluate the application of new technologies and methods developed during the project. In particular, the installation of the instrumented anchorage will be an opportunity for on-site validation of both the dissipative device and monitoring system developed within the framework of WP6. The output of the monitoring activity will in deed feed in the final design of the anchoring system in WP6.

2.12.16.2 Foreseen opportunities for application of new approaches and technologies

This case study can be considered for the application of a knowledge-based methodology developed during the project, which consists in the implementation of a step-by-step procedure or incremental approach. The interventions techniques or the intervention scheme must allow for progressive decision-making and implementation of intervention measures. The final decision is substantially supported by the progressively increasing knowledge of the real response of the building acquired before, during and after any step, by appropriately studying the real case in its real environmental conditions and conditions of use.
Another important aspect concerns the definition of guidelines and criteria for the reconstruction of the damaged parts of the building, indicating the most appropriate strengthening interventions on vertical and horizontal elements according to the techniques tested in the project.

### 2.12.16.3 Foreseen opportunities for testing and validation of proposed intervention methods

A tentative intervention proposal, aimed at seismic improvement, will be formulated during the project. The intervention proposal will take into account the application of methodologies and criteria resulting from the project. The possibility of alternative solutions, based on innovative technologies will be carefully examined. One of more alternative solutions, based on the project outcome, will be developed into detail for its possible real implementation.

The strengthening intervention may be actually implemented during the project duration (depending on several factors). In this case, the presented case study will also offer an interesting possibility for the evaluation and validation for the proposed intervention method.

### 2.12.16.4 Potential contribution to validate the project's methods and criteria

Given the fact that the two buildings have been already investigated by means of some NDT techniques, the focus of further researches should be placed on static and dynamic monitoring technologies oriented to model updating and dynamic characterization.

The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

### 2.12.16.5 Other specific opportunities provided by the proposed case study

The case study of the church of S. Biagio and the oratory of S. Giuseppe offers an interesting case regarding the study of the performance, the seismic assessment and the design of strengthening interventions in a building strongly damaged by an earthquake in a post seismic emergency scenario.

It could become a pilot project for the implementation of a standardized procedure in the reconstruction process of the building belonging to cultural heritage in the L’Aquila city centre and in the surrounding historical centres hit by the earthquake.

Table 2.28 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Activities to be developed within the project</th>
<th>Already developed or on-going activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Inspection (specify methods and technologies)</td>
<td>General geometric survey available</td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>Stone and mortar properties identified</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>based on samples taken from the building. Physical properties of mortar and stone, and mechanical properties of stone will be identified (POLIMI)</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT already performed consisting of sonic pulse velocity tests, radar tests, thermographic tests, single and double flat jack tests, dynamic identification tests</td>
</tr>
<tr>
<td></td>
<td>No additional NDT and MDT are foreseen at the moment.</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
</tr>
<tr>
<td>------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>A continuous static monitoring system is currently being installed including temperature and humidity sensors and linear displacement transducers on the main cracks. The static system will be kept active during the project duration.</td>
</tr>
<tr>
<td>2.2. Dynamic monitoring</td>
<td>A continuous dynamic monitoring system is currently being installed including 6 high sensitivity piezoelectric accelerometers. The dynamic system will be kept active during the project duration.</td>
</tr>
<tr>
<td>2.3. Monitoring by instrumented anchorage</td>
<td>An instrumented anchorage developed in WP6 is going to be installed in correspondence of the crack between façade and east-side wall of the oratory.</td>
</tr>
<tr>
<td>3. Structural analysis</td>
<td></td>
</tr>
<tr>
<td>3.1 Modelling and model updating</td>
<td>FE models available</td>
</tr>
<tr>
<td>3.2 Seismic analysis</td>
<td>Need to calibrate the models on the results of investigations and monitoring. Cross-correlation between data recorded by the three monitoring systems will be used to validate and amend the design of the instrumented anchorage and develop correlation laws so as to allow for the full characterisation of structural connections by the instrumented anchorage.</td>
</tr>
<tr>
<td>4. Intervention</td>
<td></td>
</tr>
<tr>
<td>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</td>
<td>Tentative characterization of seismic strengthening needs based on minimal intervention. Seismic strengthening proposal to be refined based on the project’s resulting methods and criteria.</td>
</tr>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td>Design of strengthening interventions regarding the substitution of the RC roof with a light timber structure; strengthening of the pillars; strengthening of vaults through the application of fibre reinforced composite materials; repair and local reconstructions of the damaged masonry structures by means of traditional techniques; insertion of steel ties.</td>
</tr>
</tbody>
</table>
4.2 Real implementation

The strengthening could be executed during the project duration, and according to the methodologies resulting from the project.

4.3 Monitoring during and after strengthening implementation

Possibility to apply incremental approach based on continuous monitoring.
Available for monitoring during and after execution of strengthening.

2.12.17 References

- Guidelines for evaluation and mitigation of seismic risk to cultural heritage with reference to technical constructions regulation, Ministry for Cultural Heritage and Activities, General Direction of Architectural Heritage and Landscape - Dept. of Civil Protection Agency (Italy)
- Modello A-DC / B-DP PCM-DPC MiBAC 2006 - Scheda per il rilievo del danno ai beni culturali - Chiese e Palazzi. source: www.protezionecivile.it
- Ramos L., Casarin F., Algeri C., Lourenço P.B., Modena C., “Investigation techniques carried out on the Qutb Minar, New Delhi, India”, Proc. 5th Int. Seminar on Structural Analysis of Historical Constructions, 2006, New Delhi, India.
2.13 VERONA ARENA - VERONA – ITALY

2.13.1 Name, location and short description

The Verona Arena is a Roman amphitheater completed around 30 AD, which is the third largest in Italy, after Rome's Colosseum and the Arena of Capua. It measures 139 meters long and 110 meters wide, and could take 25,000 spectators in its 44 tiers of marble seats. The “ludi” (shows and gladiator games) performed within its walls were so famous that they attracted spectators from far beyond the city. The current two-story façade is actually the internal support for the tiers; The round façade of the building was originally composed of white and pink limestone from Valpolicella; but after a major earthquake in 1117, which almost completely destroyed the structure's outer ring, except for the so-called "ala", the stone was quarried for re-use in other buildings. Only a fragment of the original outer perimeter wall remains. The interior is very impressive and it is used even today for public events, fairs, theatre and open-air opera during warm summer nights.
The first interventions to recover the arena's function as a theatre began during the Renaissance. Some operatic performances were later mounted in the building during the 1850s, owing to its outstanding acoustics.
2.13.2 Historical note

Between the ancient and medieval walls of the city of Verona there is an open area popularly known as the Brà. The term derives from ancient Germanic *breit* meaning broad. It is here that the Amphitheatre stands. This is without doubt the biggest and best-preserved Roman monument in an absolute sense. It is more commonly known as Arena. This was where gladiator would fight and where all circus and equestrian events were held.

The amphitheater or arena is, among the Verona's Roman monuments, without doubt the most impressive and complex architectural work, recognized as a sign and symbol of the city. It was built between the second and the third decade of the first century AD and it was erected outside the walls of the late Republican wall of the city. Its major axis is roughly parallel to the great “cardo”, probably due to reasons of planning consistency. The area on which the amphitheater is located is slightly lower compared to the current level of the surrounding soil, but if it is considered in the context of the overall levels of the Roman city, it is clear that it was chosen, or perhaps created a slight elevation of the land, i.e. a small hill.

Actually, today we can see only a part of the original amphitheater. The original edifice was built up into three orders of marble arches over thirty meters high. On top on this there was also a crowning wall with square windows, which can still be seen in some etchings by Palladio.

The Arena of Verona is third in size of its type, coming after the Colosseum and the Amphitheater of Santa Maria Capua Vetere. It is, however, certainly the most interesting due to the outstanding state of conservation of the cavea. This honor derives from the careful choice of building materials and the solid nature of the load-bearing structures. Indeed other amphitheatres, such as those in Padua and Rimini, were built in brick. Furthermore, though constantly being used, throughout the centuries the Arena has been also the object of constant care.

2.13.3 Past transformations and interventions

One of its first and most illustrious restorers was king Theodoric. In 1222, Rizzardo, count of S. Bonifacio and podestà or political leader of the Free Municipality of Verona, donated his own money for repairs. In 1228 certain communal statutes introduced a tax of 200 lira per year towards the maintenance of the Arena. It was under the Republic of Venice that the four arches, which had collapsed in the sixteenth century, were then rebuilt. The Arena, much like the Colosseum, was also, however, considered a handy quarry for ready-made blocks of stone for other walls and
palaces. Fortunately this did not create much damage overall because generally only the stone fallen from the top wing were reused elsewhere. These pieces had fallen because of the passage of time and/or earthquake. They still bear witness to their origins, however thanks to the inscriptions then made of them.

There are also other reasons for the good state of Arena. One such reason is the perfect system of hydraulic drainage. Within this “mountain of stone” the system perfectly gathers and leads away any water filtering down into it through internal veins or the vomitoria. Each of the three ring corridors and the base of the external platform have an underground drainage system running. The gallery is crossed by two other drains which lead to the outside. Underneath the intersection of these two drains there is an area set aside 34 m long, 8.85 m wide and 2 m deep, originally covered in stone slab.

In 1579 five arches of the Arena collapsed into Piazza Bra and the reconstruction was completed in 1595. At this time the arena was sometimes used as a space for tournaments and other public shows.

Was rebuilt in the '600 most of the tier, works greatly criticized in the following century, as had been altered the slope of compromising the internal configuration of the monument and the ability to understand the primitive form of the auditorium.

2.13.4 Geometrical, structural and material features

The Arena was articulated into four concentric, elliptic rings whose internal axes were respectively 73 and 44 meters and whose external axes, including the outermost wall which has almost disappeared altogether measure 152 and 123 meters in length. The 44 marble tiers gave rise to a 40-meter mass supported by arches and walls placed on the four rings. The wing remaining to testify to the overall grandeur in ancient times still boasts its four arches per level, but originally it must have had 73 pillars built out of so-called Veronese stone. This special stone came from the area around S. Ambrogio in Valpolicella and was systematically used for gates and other monuments datable in the first century AD. Every pillar was square with 2 meters wide sides. Together they formed 72 arched openings on every level which each had a 4,4 m wide corridor running around the second ring which is now openly visible.

Though dressed in stone, the ring was built up of a hardy conglomerate called “opus coementicium” composed of cement, river pebbles and brick or roof tile fragments. Theses building materials would be poured into giant moulds so as to build up the vaults, arches and walls. Huge blocks of stone would then be placed against the structure as in the “titanic walls”. Another corridor 3 m wide gives rise to the fourth, innermost ring entirely built of brick 3,6 m thick. By looking through any one of the 72 arched openings it is possible to behold the prospective fugue lines running through ever-smaller pillar and vaults right to the farthest openings composed of two
vertical monoliths and a horizontal one acting as architrave. These then lead the eye to the innermost ring, which opens up, into the gallery or pit and cavea. Here there are ramps of internal stairs leading to the wide tiers in the middle of the openings called vomitoria. On the whole, the Arena is an exceptionally solid structure. Built entirely of stone and brick, it has been capable for almost twenty centuries of supporting the enormous marble basin. This typically Roman structure is, therefore, like the Colosseum and the Provencal amphitheaters of Mines and Arles, not to mention those of Frejus, Pola and Aosta.

2.13.5 Present and foreseen future use. People at risk.

The Verona Arena is one of the most famous monument in Italy. Thousands of people visit it each year and it is also used for public events such as operas and concerts.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterized by different coefficients of use (Table 2.27).

Table 2.29 - Classes and coefficients of use CU for buildings.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use C_U</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The building is classified in the 3rd class of use, with a coefficient of use C_U = 1.5.

Its nominal life, according to the type of construction, is defined as V_N ≥ 50 years. The reference period for the seismic action is given by the product C_U x V_N = 75 years.

2.13.6 Considerations on valuable cultural content

To evaluation of the cultural content will be performed within the project.

The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:

- Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
- Categories of use (occasional or unused, frequent and very frequent)

The combination of these categories leads to define the exceeding probability (P) of the seismic action in 50 years and the importance factors γ_i for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.13.7 Present condition and damage

2.13.8 Information on local seismicity
2.13.8.1 Local seismicity
Verona is moderately seismic zone characterized by peak ground accelerations of 0.157g for a 475-years return period according to the Italian code Norme Tecniche per le Costruzioni Ministerial Decree 14/01/2008.

2.13.9 Previous and on-going studies

2.13.9.1 Introduction
Inspection works will be performed during the NIKER project, focusing in particular on the application of NDT, in particular dynamic identification tests for the dynamic characterization of the structure in order to design and implement a structural health monitoring system.

2.13.9.2 Inspection works

2.13.9.2.1 Geometric survey
A detailed geometric survey of the entire building is available.

2.13.9.2.2 Visual inspection
Visual inspections have been carried out in the past for the purpose of identifying morphology, materials and damage.

2.13.9.2.3 Deep inspection by means of NDT and MDT
The characterization of the structures of the Arena was performed by:

- Mechanical horizontal continuous coring to determine the geometry and the presence of any voids in the piers;
- Flat jack tests to determine the state of stress in the piers and to estimate the influence of the pre-stressed steel cables inserted in the piers.
- Dynamic identification tests

Core samples revealed that the so-called construction technique “anasthirosis” has been used in the construction of masonry structures, at least at the second order: the blocks, installed without mortar, have a concave internal part in order to distribute the load uniformly on the perimetric edges. This procedure was probably adopted to facilitate the stability of the pillar and to prevent the formation of tensile stresses in the block. However, this technique reduces substantially also the cross section, estimated between 50% and 60%, which creates a strong increase (about double) of the stress values compared to the full section.

Figure 2.256 - Construction technique “anasthirosis” applied for the construction of the masonry structures.
2.13.9.3 Structural analysis
Global and local FE models of the building are available. Structural analyses have been already performed including both linear and non-linear analyses, dead loading and seismic performance.

In particular a global FE model and a local FE model of the “Ala” has been implemented: natural frequency analysis and non-linear static pushover analysis have been performed (Figure 2.258 and Figure 2.259).

Figure 2.257 - Dynamic identification of the “Ala”.

![Figure 2.258 - Natural frequency analysis of the entire structure.](image)

Figure 2.258 - Natural frequency analysis of the entire structure.

![Figure 2.259 - Natural frequency analysis of the “Ala”-](image)

Figure 2.259 - Natural frequency analysis of the “Ala”-
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

Knowledge based assessment vaults D9.1 218

Figure 2.260 - Pushover analysis and seismic performance of the “Ala”.

The Italian code proposes a specific method for the seismic evaluation of the existing masonry buildings. This method is based on the failure mechanisms observed in masonry buildings after severe seismic events, and it is based on the evaluation of the limit analysis of masonry portions - considered as rigid blocks - subjected to their self weight (stabilizing effect) and horizontal forces (earthquake action). Also this method has been applied in the seismic assessment of the “Ala” of the Arena.

Figure 2.261 - Limit analysis method applied to the “Ala”.

2.13.10 Previous or on-going restoration works

2.13.10.1 General restoration works

Maintenance, involving the repair of cracks and other damage, is carried out in a continuous base. However, no consideration on the type of needed repairs, regarding in particular the seismic performance of the building, has been so far undertaken in order to re-define and improve the maintenance practices.
2.13.11 Local seismicity and characterization of seismic action

The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the building is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building’s location.

2.13.12 Proposed (additional) inspection works

2.13.12.1 Geometric survey. Aims, methods and technologies

It is necessary to perform a complete and precise geometric survey of the building, using also new technologies developed during the project in order to enrich the available geometric information.

2.13.12.2 Deep inspection by means of NDT and MDT. Aims, methods and technologies

Within the NIKER project it will be possible to perform an extensive investigations’ campaign in order to increase the knowledge level of the building and its global and local structural behavior. Deep inspections including NDT are foreseen, focusing in particular on the dynamic behavior of the structure that will be investigated by means of dynamic identification tests. The main objective is to concentrate the research activities on the dynamic survey of the building using ambient vibration tests. These tests are necessary to design and install the proposed monitoring systems.

2.13.13 Proposed monitoring

2.13.13.1 Static monitoring

A static monitoring will be installed in the Arena during the project including devices to measure a set of displacements and strains at critical points of the buildings.

20 wireless displacement transducers and 4 relative humidity and temperature sensors compose the static monitoring system.

Given its significance, particular attention will be given to the monitoring of the variation of temperature in the building during the project duration. Temperature influences largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the building is necessary for a correct interpretation and post-processing of both results.

The static monitoring will be kept active during the entire duration of the project.

2.13.13.2 Dynamic monitoring

The proposed dynamic monitoring is composed by 16 high-resolution high sensitivity acceleration transducers connected to an acquisition unit with a Wi-Fi router for remote data transmission. The positioning of the acceleration sensors will be designed and decided on the results of the structural dynamic identification.

High-density (100 sps) dynamic information will be continuously recorded. This continuous dynamic monitoring has several connected purposes: (1) Characterize the dynamic response for ambient vibration along with its dependence with environmental parameters (temperature, humidity); (2) Capture the dynamic response in the occasion of possible seismic events.
2.13.13.3 Monitoring phases

2.13.13.3.1 Monitoring before, during and after the interventions

Structural strengthening interventions are not foreseen at the moment. The installation of the monitoring system is a key activity to increase the knowledge on the structural behaviour of the monument and thus have deeper insight on its conditions. Following an integrated approach, structural health monitoring can work also as an early warning system allowing to intervene with more confidence, and only if really needed, respecting the important principle of the minimum intervention.

2.13.13.4 Modeling and structural analysis methods

It will be necessary to improve and implement the existing FE models of the Arena and to calibrate them on the results of investigations and monitoring in order to perform advanced linear and non-linear FE structural analysis.

Global and local models of the structure will be used for carrying out a detailed seismic assessment of the building. Non-linear analysis methods with appropriate constitutive models for the description of the masonry response will be used to characterize the structural response for a variety of actions and, particularly, for seismic assessment. Structural analysis will be carried out using continuous FEM macromodelling approaches involving the entire structure (global model). Local analysis will be implemented by means of the kinematic limit analysis method.

2.13.13.5 Model updating and monitoring. Interaction with monitoring

It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general layout of static and dynamic monitoring.

2.13.14 Potential contribution to the project
2.13.14.1 Compliance with project objectives
The proposed case study gives the opportunity to:
- Prove the reliability of the adopted models for the seismic assessment and the structural analysis
- Evaluate the application of new technologies and methods developed during the project.
- Use the SHM as an integrated tool to improve the knowledge of the building following an integrated approach in which it can also work as an early warning system.

2.13.14.2 Foreseen opportunities for application of new approaches and technologies
The building can be considered for the application of new technologies in particular in the field of the SHM by means of the introduction of the wireless technology. Significant effort will be carried out on the characterization of the structural response by means of both static and dynamic monitoring, and the use of the monitoring output for model updating and validation. Technologies for dynamic monitoring (as described in section 6.2) and for structural analysis will be tested and evaluated. Particular attention will be given to the combined use of monitoring and structural analysis through an integrated methodology allowing sound validation of structural models.

2.13.14.3 Potential contribution to validate the project’s methods and criteria
The focus of further researches should be placed on static and dynamic monitoring technologies oriented to model updating and dynamic characterization.

The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

Table 2.30 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection (specify methods and technologies)</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric survey</td>
<td>Complete and precise geometric survey of the building available</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Not expected within the project</td>
</tr>
<tr>
<td>1.3 Internal morphology</td>
<td>NDT and MDT already performed consisting of core sampling, flat jack tests and dynamic identification tests</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td>NDT consisting dynamic identification tests. The research effort will concentrate in particular on the dynamic survey of the building.</td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>A continuous static monitoring system is currently being installed including 20 wireless displacement transducers and 4 relative humidity and temperature sensors.</td>
</tr>
<tr>
<td>2.2. Dynamic monitoring</td>
<td>A continuous dynamic monitoring system is currently being installed including 16 high resolution high sensitivity acceleration transducers</td>
</tr>
<tr>
<td>3. Structural analysis</td>
<td></td>
</tr>
</tbody>
</table>
### 3.1 Modeling and model updating
Global and local FE models available. FE models improvement and implementation. Need to calibrate the models on the results of investigations and monitoring. Model updating techniques exploiting the monitoring results.

### 3.2 Seismic analysis
Natural frequency analysis of the entire structure and of the “Ala”. Non-linear static (pushover) analysis of the “Ala”. Kinematic analysis of the “Ala”. To be improved.

### 4. Intervention

#### 4.1 Characterization of needs for repair, maintenance and (seismic) strengthening
Structural strengthening interventions are not foreseen.

#### 4.1 Design of strengthening intervention
Structural strengthening interventions are not foreseen.

#### 4.2 Real implementation
Structural strengthening interventions are not foreseen.

#### 4.3 Monitoring during and after strengthening implementation
Possibility to apply incremental approach based on continuous monitoring. Use the SHM as an integrated tool to improve the knowledge of the building following an integrated approach in which it can also work as an early warning system.

### 2.13.15 References

- Geotecnica Veneta (2002), Indagine georcheologica e geofisica mediante metodologia georadar volta alla definizione delle fondazioni degli arcavoli dell’Arena di Verona.
2.14 THE STONE TOMB OF CANSIGNORIO DELLA SCALA - VERONA - ITALY

2.14.1 Name, location and short description

The Scaliger Tombs (in Italian: Arche scaligere) is a group of five Gothic funerary monuments in Verona, Italy, celebrating the Scaliger family, who ruled in Verona from the 13th to the late 14th century.

The tombs are located in the city center of city in a court of the church of Santa Maria Antica, separated from the street by a wall with iron grilles. Built in Gothic style, they are a series of tombs, most of which are in the shape of a small temple and covered by a baldachin. According to the French historian Georges Duby, they are one of the most outstanding examples of Gothic art.

The tombs are placed within a wrought iron enclosure decorated with a stair motif, in reference to the Italian meaning of the name of the family, della Scala.

2.14.2 Historical note

The “Scaligeri” or “della Scala” family was a dynasty that ruled Verona, in Italy, for over a century, from 1262 to 1387. Cansignorio della Scala (1340-1375) managed the city in a relatively peaceful period, and adorned Verona in a way to make it call “marmorina” (marbled) for the abundant use of ancient marbles and roman statues.

The stone tomb of Cansignorio della Scala (Figure 2.263) was built between 1374 and 1376, by will of the same Cansignorio, when he was still alive. The tomb was erected close by the St. Maria Antica church, where the tombs of Cangrande and Mastino the 2nd (his grandfather and father respectively) were already built by local workers. Differing from his ancestors, Cansignorio desired...
a monumental tomb, where the architectural aspect was more important than the decorative. The work was then commissioned to Bonino da Campione, a famous master of gothic sculpture. The monument, based on a hexagonal plan, is adorned with sculptures and spired tabernacles, with the overhanging equestrian statue of Cansignorio.

Figure 2.263 - The stone tomb of Cansignorio (on the right) and Mastino the 2nd (rear left), near the St. Maria Antica church.

Figure 2.264 - The upper part of the Cansignorio stone tomb with the equestrian statue.

2.14.3 Past transformations and interventions

Throughout the centuries, several repair interventions were necessary to preserve the delicate structure of the stone tomb, such as those carried out in the XVII, XIX and XX centuries. In 1676 the Verona municipality adopted a resolution to execute restoration works on the tomb, comporting strengthening interventions and substitutions on the upper part of the monument, without however intervening on the supporting elements.

Between 1827 and 1829 other restoration works were carried out, raising arguments on the type of marble to be used in substitutions of the deteriorated parts. Between 1838 and 1844 the fence was restored, and on the 24th of July 1840 a portion of the southern gablet fell down, being subsequently restored (1846) and lodged back in the original position. Substitutions comported the use of Candoglia marble elements, secured with iron clamps fixed with melted lead. The sealing of the cracks was performed with filler. Main interventions carried out were: the reconstruction of the spires of some tabernacles; the positioning of steel reinforcing elements on two columns of a tabernacle; the complete reconstruction of a column and capital of a tabernacle, and of some gablets between the spires; the reconstruction of the tail and the left rear leg of the horse in the equestrian statue; the substitution of the copper tie beams of the tabernacles with saintwarriors with new ones in iron; the sealing of the vault’s groins.

Other interventions, similar to those executed at the half of the XIX century, were carried out between 1910 and 1914. The monument was then protected against bombing during the two world
In 1919, after the removal of the shields, some light restorations were carried out. Then, during the positioning of the shields of the 2nd world war, an analysis of the conditions of the tombs was carried out, with successive light restoration works.

### 2.14.4 Geometrical, structural and material features

A detailed laser scanning and photogrammetric survey of the entire structure is available. The tomb is surrounded by an hexagonal wrought iron fence, at whose corners rise six pillars sustaining gothic tabernacles, containing statues of the saint-warriors (St. George, St. Martin, St. Quirinus, St. Sigismund, St. Valentine and St. Louis, king of France). The tomb starts with six columns sustaining a red marble slab on which finds place the white marble sarcophagus, sustained by eight pillars and decorated with bas-reliefs representing Gospel scenes. The cover of the sarcophagus hosts a lying statue of Cansignorio, watched over by angels.

At the second level, six further spiral columns sustain the canopy with polyleobed arches. Above these finds place a cornice sustaining six gablets with allegorical figures representing the virtues. At the corners are positioned six further tabernacles with statues of angels. The roof, corresponding to an hexagonal pyramid made of white marble, finally supports the massive equestrian statue of Cansignorio (Figure 2.264).

The stones used for the erection of the tomb are the “Candoglia” white marble, the same employed in the Milan’s cathedral, and the “Rosso di Verona” (Verona’s red marble), besides the Pietra Gallina (a soft limestone from Vicenza). The inner part of the roof (above the crossed vault and behind the stone facing of the canopy) is composed by solid brickwork masonry.

![Figure 2.265 - Plan view of the tomb at different heights.](image-url)
2.14.5 Present and foreseen future use. People at risk.

According to the Italian code NTC 2008 buildings are classified by class of use, depending on the consequences on the collapse. The classes of use are characterised by different coefficients of use (Table 2.27).

Table 2.31 - Classes and coefficients of use $C_U$ for buildings.

<table>
<thead>
<tr>
<th>Classes of use</th>
<th>Buildings</th>
<th>Coefficient of use $C_U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
<td>0.7</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging to the other categories</td>
<td>1.0</td>
</tr>
<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, museums,...</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The stone tomb of Cansignorio is classified in the 3rd class of use, with a coefficient of use $C_U = 1.5$. Its nominal life, according to the type of construction, is defined as $V_N \geq 50$ years. The reference period for the seismic action is given by the product $C_U \times V_N = 75$ years.

2.14.6 Considerations on valuable cultural content

See §1.4 for a detailed description of the valuable cultural content of the tomb. The Guidelines for evaluation and mitigation of seismic risk to cultural heritage (July 2006) of the Ministry of Cultural Heritage define different levels of seismic protection for CH buildings according to the following categories:
Categories of relevance (limited, medium and high), defined on the basis of the knowledge of the building
Categories of use (occasional or unused, frequent and very frequent)

The combination of these categories leads to define the exceeding probability (P) of the seismic action in 50 years and the importance factors $\gamma_I$ for each CH building. These parameters are essential for the definition of the seismic action for the seismic verifications in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

2.14.7 Present condition and damage

2.14.7.1 Considerations on the present condition
In general the stone tomb does not present indications of worrying structural problems.

2.14.7.2 Main observed damage and decay
Widespread cracks were noted in the cornice above the pointed arches, and in the same arches close by their keystones (Figure 2.267a).

Parts of the cornices tended to separate, although this is not a recent damaging process, since iron clamps of previous interventions were found. The existing iron tie beams, spanning between the capitals of the upper order columns are locally damaged by oxidation.

The iron tie beams connecting the tabernacles of the fence to the spiral columns, likely positioned during the XIX c. interventions, manifested marked decay at the connection with the original copper tie beams anchored to the stone.

The removal of the copper “bandage” provided to the equestrian statue of Cansignorio during past interventions for the strengthening of the supports of the statue, highlighted the presence of material decay, with severe cracks and voids (Fig. 12a, b).

The expansion of the iron tie beams due to material oxidation caused the cracking of the capitals of the upper order columns, in some cases with severe effects.
2.14.8 Information on local seismicity

2.14.8.1 Local seismicity

Verona is moderately seismic zone characterized by peak ground accelerations of 0.157g for a 475-years return period according to the Italian code Norme Tecniche per le Costruzioni Ministerial Decree 14/01/2008.

2.14.9 Previous and on-going studies

2.14.9.1 Introduction

Previous detailed studies involving inspection, monitoring and structural analyses have been carried out since 2006.

2.14.9.2 Inspection works

2.14.9.2.1 Geometric survey

A detailed laser scanning and photogrammetric survey of the entire structure is available.

2.14.9.2.2 Visual inspection

Detailed visual inspection has been carried out on the entire structure for the purpose of identifying morphology, materials and damage. Cracking and damage has been mapped in detail.

2.14.9.2.3 Deep inspection by means of NDT and MDT

Deep inspection was carried out by means of dynamic identification tests performed in August 2006. The investigation campaign were aimed the definition of the optimal SHM (Structural Health Monitoring) system sensors’ positioning, and at the characterization of the dynamic properties of the monument for FE modelling calibration purposes.

Following the mode shapes emerged from the FE numerical model, sensors were placed at the first level (in the stone slab where the sarcophagus stands), at the second level (on the cornice above the pointed arches) and at the top of the monument (at the foot of the equestrian statue). A total of six sensors was employed, considering three test setups for a total of 6 acquisition points, recording the acceleration in orthogonal (and parallel to the ground) directions.

A compact unit provided with 24-bit digital acquisition cards, connected to piezoelectric mono axial acceleration transducers, composes the acquisition system. Once fixed the transducers to the structure in the selected positions, tests consisted in acquiring data over a predetermined period, at a determinate sample rate. Each test setup consisted in recording the signal two times (65'536 points each) with a sampling frequency of 100 SPS (samples per second), with an overall setup signal recording duration of 21'511”. For the identification of the modal parameters (natural frequencies and corresponding mode shapes), output only identification techniques were used (Operational Modal Analysis). In particular, the recorded ambient vibrations were related to the wind excitation and urban traffic.

2.14.9.3 Monitoring

The Structural Health Monitoring System (installed in December 2006) is aimed at the control of static and dynamic parameters related to the structural functioning of the monument. The system is composed by:

1 acquisition unit;
6 piezoelectric accelerometers;
2 potentiometric displacement transducers;
1 temperature and relative humidity sensor.

The central unit, located at the base of the tomb, is provided with a Wi-Fi router for remote data transmission.
The monitoring strategy is conceived both to collect data at predetermined time-intervals (periodic monitoring, i.e. cracks opening, changes in the dynamic response) and to automatically start to save data in case of significant external events (such as seismic events). Such controls will permit to appreciate possible variations in the assessed structural functioning with the passing of time and to have a record of the dynamic behavior of the stone tomb during severe events.

The acceleration transducers are placed in suitable positions in relation to the mode shapes of the structure, as shown by the numerical modeling/dynamic identification. Four sensors are placed on two levels for the evaluation of the vibration in the NS and EW direction (bending modes) and in the horizontal planes (torsion modes).

A couple of reference sensors is fixed at the base for the record of the ground acceleration both in operational conditions (i.e. evaluation of the traffic induced vibrations) and during seismic events. A temperature/relative humidity sensor is fixed at the intrados of the marble slab (first level). The displacement transducers are positioned across significant cracks. The temperature, relative humidity and displacement of the selected points (crack mouth opening) are recorded each 6 hours, corresponding to 4 daily readings. Dynamic data are collected both at fixed time intervals (each 48 hours, approximately 22' of recording at a sample rate of 100 Hz) and on a trigger basis (shorter records, signals are recorded when the vibration exceeds a predefined threshold).

2.14.9.4 Structural analysis

A detailed FE numerical model, based on a laser scanner geometrical survey of the monument previously carried out, was implemented in order to evaluate the static and dynamic behaviour of the monument. The evaluation of the initial results of the numerical model (linear static and natural frequency analyses) assisted the design phase of the strengthening intervention and indicated the most suitable places for the sensors’ positioning (dynamic identification and monitoring). The first model was calibrated on the basis of the results of the experimental activities, in order to be subsequently used to simulate the response of the monument to different external actions.

Advanced non-linear FEM analyses were performed including non linear static (pushover) and non linear dynamic analyses.

Figure 2.268 - Rendered view of the FE model, East side and corresponding mesh (a). First six mode shapes of the numerical model (b).
2.14.10 Previous or on-going restoration works

2.14.10.1 General restoration works
In parallel with the studies previously reported, and benefiting from their outcomes, a light strengthening intervention was carried out. New structural elements were introduced as precautionary measures, e.g. by providing redundant confining systems, to collaborate with existing deteriorated elements and acting in case of sudden structural deficiency of the original material.

In general the stone tomb does not present indications of worrying structural problems. Interventions mainly consisted in hooping the monument at different levels. With reference to Figure 8, interventions included: A) hooping the base of the canopy with a stainless steel cable; B) hooping the capitals with a couple of stainless steel cables; C) repair of the junctions of existing tie beams; D) hooping the tabernacles with high resistance stainless steel cable. Local interventions consisted in: (1) binding the damaged supports (horse’s hooves) of the equestrian statue of Cansignorio by means of CFRP strips; (2) strengthening a cracked capital with hoopings in high resistance stainless steel cable.

Figure 2.269 - Strengthening of the stone tomb of Cansignorio della Scala, detail of the interventions.

2.14.11 Historical research
Extensive historical researches have been already undertaken. No additional studies are expected within the project.

2.14.12 Local seismicity and characterization of seismic action
The possibility of carrying out additional investigation for a better characterization of the seismic action for the site where the building is located is not disregarded in order to determine a demand spectra adequate for the corresponding site, taking into account the local geological and geotechnical features of the building’s location.

2.14.13 Proposed (additional) inspection works
2.14.13.1 Geometric survey. Aims, methods and technologies
A detailed geometric survey is already available, including laser scanning and photogrammetric surveys. No additional surveys are expected.

2.14.14 Proposed monitoring
The SHM system is composed by:
1 acquisition unit;
6 piezoelectric accelerometers;
2 potentiometric displacement transducers;
1 temperature and relative humidity sensor.
It will be kept active during the project duration in order to implement a step-by-step approach for interventions (incremental approach).

2.14.14.1 Monitoring phases
2.14.14.1.1 Monitoring before intervention
The static and dynamic monitoring systems were active before the execution of the strengthening interventions and restoration works that have been already performed.

2.14.14.1.2 Monitoring and control during intervention
The static and dynamic monitoring systems were active during the execution of the strengthening interventions and restoration works.

2.14.14.1.3 Monitoring and survey after intervention
It is intended to have both the static and dynamic monitoring system active during a period of, at least, two years after the implementation of the strengthening interventions. In case of envisaging and applying a possible step-by-step approach for intervention (incremental approach), static and dynamic monitoring could also be considered for a longer period according to the needs of the applied procedure.

2.14.15 Proposed structural analysis. Aims and methods
2.14.15.1 Modeling and structural analysis methods
No additional modeling and structural analyses are considered.

2.14.15.2 Model updating and monitoring. Interaction with monitoring
It is intended to devote significant effort in model updating and validation by means of intensive use of monitoring results. Numerical simulation using the models made available will be used for taking decisions on the characteristics and the general layout of static and dynamic monitoring.

2.14.15.3 Assessment of possible intervention methods
Structural models, using the same modeling approaches and tools described in section §6.4, will be also utilized in order to simulate the effectiveness of the implemented strengthening techniques.

2.14.16 Potential contribution to the project
2.14.16.1 Compliance with project objectives
The proposed case study gives the opportunity to:
- Prove the reliability of the adopted models for the seismic assessment and the structural analysis
- Verify the effectiveness of the proposed structural solutions.
- Evaluate the application of new technologies and methods developed during the project.

### 2.14.16.2 Foreseen opportunities for application of new approaches and technologies

Significant effort will be carried out on the characterization of the structural response by means of both static and dynamic monitoring, and the use of the monitoring output for model updating and validation. Technologies for dynamic monitoring and for structural analysis will be tested and evaluated. Particular attention will be given to the combined use of monitoring and structural analysis through an integrated methodology allowing sound validation of structural models.

### 2.14.16.3 Foreseen opportunities for testing and validation of proposed intervention methods

The strengthening intervention has been already implemented. The presented case study will also offer an interesting possibility for the evaluation and validation for the applied intervention method.

### 2.14.16.4 Potential contribution to validate the project's methods and criteria

The building can be considered as well for the selection, design, and application of minimized interventions and for the evaluation intervention strategies, including possible step-by-step approaches. It is expected that the presented case study will contribute very significantly to the calibration and validation of the general methodology, tools and criteria resulting from the project thanks to the opportunities it offers for their real implementation.

Table 2.32 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1- Inspection</strong>&lt;br&gt;(specify methods and technologies)&lt;br&gt;1.1 Geometric survey&lt;br&gt;General laser scanning and photogrammetric geometric survey available&lt;br&gt;1.2 Materials&lt;br&gt;Not performed&lt;br&gt;1.3 Internal morphology&lt;br&gt;NDT consisting of dynamic identification tests.</td>
<td><strong>Activities to be developed within the project</strong>&lt;br&gt;2. Monitoring&lt;br&gt;2.1 Static monitoring&lt;br&gt;4 years of continuous static monitoring already available including temperature and humidity sensors and 2 potentiometric displacement transducers&lt;br&gt;2.2. Dynamic monitoring&lt;br&gt;4 years of continuous dynamic monitoring already available, including 6 high sensitivity piezoelectric accelerometers.</td>
</tr>
</tbody>
</table>
4. Intervention

| 4.1 Characterization of needs for repair, maintenance and (seismic) strengthening | Strengthening interventions already designed and implemented |
| 4.1 Design of strengthening intervention | Design of strengthening interventions and seismic retrofitting already performed. |
| 4.2 Real implementation | Strengthening interventions already implemented. |
| 4.3 Monitoring during and after strengthening implementation | Possibility to apply incremental approach and step-by-step procedure based on continuous monitoring. |

2.14.17 Reference

- Bertolini V. Da Nantes a Brou e a Ginevra: influssi possibili o irrefutabili delle Arche Scaligere. 1993, Verona (in Italian)
- De Maffei F. Le arche scaligere. 1955, Verona (in Italian)
- Ramos L., Casarin F., Algeri C., Lourenço P.B., Modena C. Investigation techniques carried out on the Qutb Minar, New Delhi, India. Proc. 5th Int. Seminar on Structural Analysis of Historical Constructions, 2006, New Delhi, India
2.15 SANTÍSSIMO SACRAMENTO MONASTERY - LISBON, PORTUGAL

2.15.1 Name, location and short description

The Santíssimo Sacramento Monastery, or Vimiosos Monastery (located in Lisbon downtown, Prazeres parish), accessed by the Sacramento to Alcântara Street, number 43 to 51, appears on the DGEMN and IMP heritage inventory. As official protection degree, is part of the “Palácio das Necessidades” Especial Protection zone. At the present time being the monastery is overlooked by IGESPAR (institute that manages architectonical and archeological heritage in Portugal), hereinafter also designated as site owner.

Due to the sizeable dimension of the complex (monastery, church and annexed buildings), we led our choice for a possible case study to the two floors and attic building, bellow located.

![Figure 2.270 - Case study location on the Sacramento Monastery Compound.](image)

2.15.2 Historical note

The referred building is contiguous to the Santíssimo Sacramento Church, and so it’s part of the Monastery history and legacy.
The first construction included a one-nave Church, finished in 1620, raised to east and with three small chantries, later demolished to be reconstructed. Few years before, the construction of the monastery took place (1612-1616), sheltering initially 35 nuns, coming from families of high nobility, number that more than doubled over the following years. In 1635, João de Vasconcelos, the new vicar of the monastery, decided to build a new church, demolishing the old one for being too tall and gri, rising another on the same place, for gallant and admirable architecture manufacture. The new church, with a Greek cross plan and a crypt under the sanctuary was located in the extreme east of the Convent set. The proposed case study, constructed near the dates of 1850-1860, and communicating with the Church by a chamber, had a residential use till the early years of the XX century, time when the property was incorporated into the Army deposit, adjacent to the building, having been adapted for office work.

2.15.3 Historical and artistic significance

The Monastery himself has cultural and artistic significance, due mainly to the pictorial and sculptural elements that can be found, mainly in the cloister and church interior. The proper building to study (annexed to the church) doesn't have special relevance to the historic and artistic context of the monastery.

2.15.3.1 Architectural arrangement, structure and materials

The proposed case study comprehends a 2 floor building with an attic, in a plant L-shape architecture. The ground floor height is of 3.2 m, while the 1st floor is 2.8 m. The attic has headroom ranging from 2.5 m to 3.2 m, while the overall building height is 11.3 m.
Regarding the structure, is composed by masonry exterior walls mainly of limestone, although we were able to find some brick stone elements, which should have appeared from some changes performed during the usage of the building. The thicknesses of these walls range from 85 to 95 cm.

A small number of the interior walls are also made of stone masonry (with some structural role), but the majority is typical front walls from the “pombalino era”, with his wooden structure in Stº André crosses. It was also detected some non structural noggin walls.

The floor diaphragms are made of wooden beams supporting a wood pavement. The types of wood identified were pine and oak.
The building cover is by a ceramic roof tile that has suffered a significant modification from its initial framing. It has a non conventional wood structure supporting the framework under the tile, emerged form a structural variation implemented to the building. It was also detected a concrete ring made to support the new roof structure, improving the attic headroom near the exterior walls. The entity that guardianships the monastery (IGESPAR) pretends to eliminate these structural incursions, returning to the original (and authentic) form.

2.15.4 Present and foreseen future use

The building at the present stage is unused, due to his inhabitance state. IGESPAR pretends to develop on this space the future CNU (National Committee for UNESCO) installations, since the present conditions for them are insufficient for their requirements.

Concerning the structural rehabilitation of the commodity, IGESPAR made notice that they will keep the original concepts for the building (architectural and structural) with light changes just to fit the best abilities to the new facilities. The time schedule for the structural intervention fits the Niker Project agenda perfectly.

The future usage represents a significant improvement to the CNU conditions as schemed at the following views:
2.15.5 Considerations on valuable cultural contents

No valuable cultural contents are present.

2.15.6 Present condition and damage

2.15.6.1 Considerations on the present condition

The structure mainly presents its original features, being a good representation of the era when the building was raised ("pombalina" construction).

2.15.6.2 Main observed alteration, damage and decay

Regarding the preservation state, it was possible to recognize some needs of intervention on the wood pieces, mainly oriented to regain the original behavior of the structural elements (some replacements may be needed).

By the local surveys already carried out, it was possible to identify some constructional flaws, regarding the connections between structural elements (wall-wall or floor-wall). No excessive load behavior was yet identified.
The structural disturbance provoked by the roof structure modification (as explained), needs a careful analysis not only because the new usages, but also to assure the proper load paths for all the building.

2.15.7 Information on local seismicity

2.15.7.1 Local seismicity
The rules relating to the action of earthquakes deserve special attention in the Portuguese legislation (Decree-Law No. 235/83), resulting in it the knowledge currently available about the distribution of seismicity in the country (according to regulatory seismic zoning map).
For quantification of the action of earthquakes is considered the country divided into four areas that by decreasing seismicity, are designated A, B, C and D (Lisbon lies in zone A).

![Portugal seismic areas](image)

Figure 2.286 - Portugal seismic areas.

2.15.7.2 Characterization of the seismic action
The seismic action revolves from a set of ground vibrations that are transmitted to the structures during the occurrence of an earthquake. The characteristic values of the action are measured according to the seismicity of the area where lies the construction, and the nature of the terrain where it is deployed:
- Terrain Type I - rocky and coherent stiff soils;
- Terrain Type II - very hard, hard and medium consistency coherent soils; compact incoherent soils;
- Terrain Type III - soft and very soft coherent soils; loose incoherent soils;

The Portuguese regulation defines as sufficient to check the structural safety of two seismic actions, one representing a moderate earthquake of magnitude to a small focal length (seismic type 1), and another earthquake of greater magnitude and a longer focal length (seismic type 2). For the horizontal axes of any system of direct and orthogonal axes, and for each type of seismic
action, the spectral power density of speed variation (acceleration) is given by the following tables (and for zone A). For the vertical axis of a similar axis system, the same spectral power density is indicated as four ninths of the same table.

\[
\begin{array}{|c|c|c|c|c|}
\hline
f (Hz) & S(f) (cm/s^2)^2/Hz \\
\hline
\hline
0.04 & 0 & 0 & 0.02 & 0 \\
1.05 & 250 & 0.9 & 220 & 0.75 & 190 \\
2.1 & 360 & 1.8 & 300 & 1.5 & 240 \\
4.2 & 360 & 3.5 & 300 & 3.0 & 240 \\
8.4 & 160 & 7.2 & 130 & 6.0 & 100 \\
16.8 & 50 & 14.4 & 40 & 12.0 & 35 \\
20.0 & 20 & 20.0 & 16 & 20.0 & 12 \\
\hline
\end{array}
\]

Figure 2.287 - speed variation spectral power density.

In the current cases, as we do not have enough information to define the action of earthquakes, we may adopt the following simplifications, which often also result in a satisfactory characterization of the action:

At each point can be assumed that the spectral densities of joint movements under any set of horizontal axes of a system of direct and orthogonal axes, are zero;

The effects of spatial variation of seismic ground motion can be quantified from an autocorrelation function, and is permissible to consider that, for each frequency band in which the movement can be broken down, this function vanishes for distances of the order of 2 to 6 wave lengths;

In the case of structures where the natural frequencies of vibration modes that contribute significantly to the response are well separated (the ratio between any two frequencies outside the range 0.67 to 1.5), the seismic action can be simplistically quantified by average response spectra. Such spectra, in relation to the horizontal components of translation are given, for zone A, in Figures III-2, III-3 and III-4. The average response spectra for the vertical component are obtained by multiplying the referred Y-axis by two thirds. Note that the quantification of seismic response spectra is only applicable, in principle, to structures which are valid the above simplifications:
2.15.8 Previous and on-going studies

A complete geometric survey was established in 2001 by the site owner (IGESPAR), resulting from it the actual set of architectural drawings.

2.15.9 Previous or on-going restoration works

2.15.9.1 General restoration works

IGESPAR promoted the restoration of one of the exterior facades, respecting the original makeup of the plaster. These works are concluded.

Figure 2.288 - response spectra (speed variation) for each type of terrain.

Figure 2.289 - facade 1 prior to the intervention.

Figure 2.290 - facade 2 prior the intervention.
2.15.9.2 Actions oriented to seismic retrofittion
No actions oriented to seismic retrofittion were carried out.

2.15.10 Local seismicity and characterization of seismic action
Concerning the local seismicity and characterization of seismic action, it is proposed to settle the rules established by the Portuguese regulation to the present case study.

2.15.11 Inspection works

2.15.11.1 Surveys and other inspection works
The existent survey has the need for an improvement regarding mainly to stability issues and materials recognition. For that we propose a diagnosis based on the following inspection and essay methods:
- Structural survey for the masonry building, including the wooden structures;
- Survey of anomalies in the masonry building, including the wooden structures;
- Evaluation of the wooden elements integrity;
- Masonry Sonic tomography;
- Tests with plan hydraulics to evaluate the stress state and mechanical characteristics of masonry structures;
- Laboratorial essays of masonry in-situ samples;

The technologies proceedings are to be presented on following Deliverables. Others surveys are to be expected regarding the development of the stability project of the intervention (for instance the study of site foundations and geology). For those it is hoped that this information will be shared by the site owner (IGESPAR).

2.15.12 Proposed structural analysis. Aims and methods

2.15.12.1 Modeling and structural analysis methods
Resulting from other WP’s, we propose to use the numerical modeling of the reinforcement techniques achieved on those WP’s, by adjusting them to the needs identified on the structural diagnosis.

This fitness will allow developing the proper dynamic behavior models of materials, structural elements, connections, and substructures, before and after the reinforcement (as well as the needs of reinforcement).

The analysis methods are to be agreed with other Niker partners.

2.15.12.2 Assessment of possible intervention methods
At the present time being it is possible to confirm the need of intervention on three major types of anomalies:
- Inefficient connection between structural elements (retrofitting developed at WP.6);
- Improper present roof structure (retrofitting developed at WP.5);
- wooden elements in disrepair (retrofitting developed at WP.4 and WP.5);

Resulting from the structural diagnosis, other needs may be presented and added to the retrofitting project.

2.15.13 Potential contribution to the project

2.15.13.1 Compliance with project objectives
Although the proposed case study doesn’t appear with a huge significance in means of cultural heritage, it still largely accounts to the most common building style in the national rehabilitation scenario (that we identify has “Pombalino” style). The proposed intervention will assess what future actions regarding building of its kind will involve, which, and having regard to its spread throughout the Mediterranean basin, should represent a handful of significant upgrade interventions, greatly increasing the seismic behavioral parameters of the buildings intervened, as previewed at the Niker purposes.

2.15.13.2 Foreseen opportunities for application of new approaches and technologies

No major opportunities are identified to apply the monitoring technologies in development by the Niker Project, due to its minor relevance to the historic and artistic context of the Portuguese architectonic heritage. This significance will permit however another kind of intervention, more extensive and profound, which must be accompanied by a monitoring process that allows follow the application of the reinforcement techniques provided, with the necessary security for such kind of intervention.

2.15.13.3 Foreseen opportunities for testing and validation of proposed intervention methods

The presented case study seems to fit a considerable structural intervention to equip the building with satisfactorily behavioral parameters to the needs of future users (Portuguese National Committee for UNESCO). It will also be an opportunity for them (and others with the same interest) to attend on-site the processes created by the Niker Project, to ensure the conservation of cultural heritage and the mitigation of seismic hazard in ancient buildings still in full use.

For MONU is also the perfect opportunity to build up some of the appliance processes for the retrofitting techniques developed by the other partners. With this we will give a major contribution to their improvement and final definition, by creating the procedures drafts to support the guidelines in development at WP.10.

2.15.13.4 Potential contribution to validate the project’s methods and criteria

The proposed case study is an important step in the development of suitable guidelines in WP.10, by ensuring that the proposed methodologies will be useful and relevant to on-site practice. Also insures that theoretical methods developed are feasible, as difficulties encountered on site might not be the same as the ones encountered in the laboratories try outs.

Table 2.33 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>Tasks already carried out or in course, with available results</th>
<th>Tasks to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Inspection</td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric and Structural Survey</td>
<td>Geometric survey carried out by IGESPAR</td>
</tr>
<tr>
<td></td>
<td>Structural Survey and Diagnosis</td>
</tr>
<tr>
<td></td>
<td>Masonry Sonic Tomography and plan</td>
</tr>
<tr>
<td></td>
<td>hydraulics essays</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>None has been carried out</td>
</tr>
<tr>
<td></td>
<td>Evaluation of wooden elements integrity</td>
</tr>
<tr>
<td></td>
<td>Laboratory testing of masonry in-situ</td>
</tr>
<tr>
<td></td>
<td>samples</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
</tr>
<tr>
<td>2.1 Static monitoring</td>
<td>None has been carried out</td>
</tr>
<tr>
<td></td>
<td>None to be carried out</td>
</tr>
<tr>
<td>2.2. Dynamic</td>
<td>None has been carried out</td>
</tr>
<tr>
<td></td>
<td>None to be carried out</td>
</tr>
</tbody>
</table>
### Monitoring

<table>
<thead>
<tr>
<th>3. Structural analysis</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>3.1 Modelling and model updating</strong></td>
<td>None has been carried out</td>
</tr>
<tr>
<td><strong>3.2 Seismic analysis</strong></td>
<td>None has been carried out</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>4. Intervention</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4.1 Characterization of needs for repair, maintenance and (seismic) strengthening</strong></td>
<td>None has been carried out</td>
</tr>
<tr>
<td><strong>4.1 Design of strengthening intervention</strong></td>
<td>None has been carried out</td>
</tr>
<tr>
<td><strong>4.2 Real implementation</strong></td>
<td>None has been carried out</td>
</tr>
<tr>
<td><strong>4.3 Monitoring during and after strengthening implementation</strong></td>
<td>None has been carried out</td>
</tr>
</tbody>
</table>
2.16 AMBEL PRECEPTORY, ZARAGOZA, SPAIN

2.16.1.1 Name, location and description
The Ambel Preceptory is a construction complex situated in the Zaragoza province of Spain, mostly built in 1.1m thick rammed earth walls. The building is dated between the 10th and the 16th century, with a number of geometrical and structural modifications and repair attempts dating back to the end of the 18th century.

Extensive studies by Dr Christopher Gerrard (Gerrard 1999; Gerrard 2003) and Dr Paul Jaquin (Jaquin 2004, Jaquin 2008) document the geometry, state of damage and crack development of the building, with focus on the four-storey granary at the north-east corner of the site (See Figure 2.291, right; room 139 in Plan on Figure 2.292). Despite the complexity of the chronology of the site, different construction and repair phases have already been researched and documented (Gerrard 2003).
2.16.2 Historical note

The site is reported by Jaquin (2008) to have been an administrative center by the Templar Order in the 12th century and to then have been occupied by the Hospitalers, also known as the Knights of Malta. The building has served different purposes over time: that of castle, monastery, palace, administration centre and farmhouse.

2.16.3 Structural and material features

Figure 2.292 - The Ambel building complex, based on plan by Jose´ Corsini (Gerrard 2003).
Structural and material features are varied, due to the building complex being constructed over centuries, and later being repaired. The historic evolution of the building is well documented (Gerrard, 2003).

The building is almost entirely made of rammed earth and adobe bricks, but parts are also constructed in stone rubble, for instance the basement, while other parts are brick, for instance the quoins of the gable wall. Due to the building, and in particular the granary, exhibiting significant structural problems, further investigation and remediation are required.

An attempt was made to arrest crack development on this part of the complex by introducing steel tie bars in the year 2000, which appear on the basis of crack monitoring to be ineffective, as crack widths continue to increase.

2.16.4 Present and foreseen future use. People at risk.

The basement today is reported by Jaquin to be a wine and storage cellar. The part of the complex exhibiting most damage is at present uninhabited.

2.16.5 Present condition and damage

2.16.5.1 Considerations on the present condition
Jaquin (2008) reports a number of structural issues being the cause of cracking especially at the north-east tower of the granary, where cracks between 6 and 9 meters in length on the exterior of the building are visible. Crack widths have been monitored since 1997 and are considerably affected by seasonal and daily temperature changes. Cracks are believed to have been caused by differential ground settlement, failure of one horizontal level, rendering decay, water ingress.

2.16.5.2 Main observed damage and decay
Main observed damage and decay affects the north-west part of the complex, and in particular the four-floor granary. The top of gable wall leans outwards, with a continuous worsening of this condition despite steel tie bars being introduced in the year 2000 to arrest this out-of-plane movement. With exception from the anchoring points, the upper part of the gable wall is therefore in structural discontinuity with perpendicular walls. The discontinuity affects the interface between the rammed earth material of the perpendicular walls and the brick masonry of the gable wall, i.e. the brick quoins. This damage is believed to have been initiated by ground settlement at the north side of the north-east tower resulting in differential settlement. The cause of the ground settlement is likely to be an ineffective retaining wall to the north of the gable wall.

The extensive cracking on the eastern wall of the granary is believed to originate from excessive point loading resulting from the horizontal elements in conjunction with lack of a timber wall plate. Other non-structural damage is the splitting of the external render protecting the rammed earth in parts of the building. The presence of plants, instead, is thought to not be affecting the ageing of the building negatively.

2.16.6 Information on local seismicity

2.16.6.1 Local seismicity
The seismic hazard of the site is low, with peak ground accelerations ranging from 0.04 to 0.08 g, with 10% probability of being exceeded in 50 years.

Table 2.34 lists the minor earthquakes documented by United States Geological Survey which have affected the area covering the 50km-radius area surrounding the site.
2.16.7 Previous and on-going studies

2.16.7.1 Inspection works

   2.16.7.1.1 Geometric survey
   Plans and elevations dating back to 1957 are available.

   2.16.7.1.2 Other inspection works
   Evolution of the building: The historic evolution of the building is well documented (Gerrard, 2003). See Figure 2.292.

2.16.7.2 Monitoring
   Crack monitoring as been carried out once or twice a year since 1997, i.e. before and after introduction of steel ties in 2000. Introduction of steel ties at ground, first and second floor levels; Repair of cracks; both carried out in the year 2000.

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10, Project Nr. 382:SESAME
2.16.7.3 Structural analysis
FEM based on 3D CAD drawing of the structure based on 1957 survey geometry. Rammed earth was modelled as a linear elastic homogeneous material. In the model wall-floor interactions and settlement were ignored. (Jaquin et al. 2004).

2.16.8 Previous or on-going restoration works

2.16.8.1 General restoration works
Previous restoration works carried out include crack filling and introduction of steel tie anchors in the year 2000 at ground, first and second floor levels. Jaquin et al. 2004 suggest alternative crack filling with suitable grout and an alternative to present anchors, i.e. Grouted anchors or nylon strapping.

2.16.9 Proposed new studies

2.16.9.1 Inspection works.

2.16.9.1.1 Geometric survey. Aims, methods and technologies
A geometrical survey aimed at the preparation of the numerical model will be carried out. This will focus on identifying material differences and vulnerabilities, as well as structural discontinuities, for instance eroded areas with considerable cross section reduction, cracks and detachments. Research will focus on the most damaged part of the building, i.e. the North East tower, which exhibits progressing damage despite repair attempts carried out in the year 2000. The detailed survey of the geometric characteristics of the North East tower will be aimed at providing reliable data for a detailed analysis of architectural characteristics and for the evaluation of the mechanical behaviour of the structure via numerical analysis. The technologies used will be laser meter and photographic recording. Thanks to the well recorded historical evolution of the building, which
already indicates different construction phases and materials (See Fig. 2 and Fig. 3 below) it will be possible to focus particular attention to the presence or absence, and type of structural connection between adjacent buildings constructed at different times, and to the connection between original fabric and repair materials. These will be complemented by results from direct inspection and NDT inspection.

Figure 2.294 - Distribution of different construction materials in the lower part of the building complex (Gerrard, 2003).

2.16.9.1.2 Research on materials. Aims, methods and technologies

Construction materials present in the building complex are numerous; to some extent their variation is dependent on the different construction phases spanning between the X and the XVI century, and the later (XVIII) reparation phases. Ashlar is reported to be the main construction material in the pre-templar building parts, whereas rammed earth, either simple or in combination with fired clay bricks, fired clay bricks, and to a relatively limited extent adobe.

As shown in Fig. 3, the north Eastern part of the building is almost entirely built in rammed earth and fired brick, at some locations interlaced whereas at other locations used independently.

Plain Rammed earth is common in the structure, and is reported by Gerrard (2003) to be composed of a very compact clay in which fragments of charcoal, vegetable matter and ceramic fragments. The walls are, however, not homogeneous: the outer lining of the walls was originally
protected by a lime-rich earthen material which is not the same nature as the inner core. Research on materials within the NIKER project will aim at defining the nature of these materials more precisely, both to know its composition and therefore assess compatibility of repair materials, and where possible to measure decay parameters (chemical and mechanical).

The presence of petrographic analyses is not a precise enough basis on which material characterisation can be based. It is therefore envisaged to carry out suitable chemical, physical and mechanical characterisation by means of laboratory testing of limited historic samples. Material composition and content for the earthen materials composing the walls will be carried out. Depending on foreseen difficulties in extracting intact samples from the historic structure, due to its brittle nature, mechanical tests to define compressive strength, poisson ratio and elastic modulus will be carried out.

Due to the foreseen difficulties in extracting wall samples, material research will initially only focus on NDT qualitative assessment of moisture content. Coring of material will also allow to carry out X-Ray diffractometry, salt (photometric determination of anion) and moisture content (Darr) testing. Due to the effect of moisture content on the compressive strength of earthen materials, an increase of the current moisture content should be avoided.

Salt tests are relevant to durability as material degradation is to be expected in any earth building materials containing extremely high concentrations of salts. High salinity leads to crystallisation processes or causes - depending on relative humidity - adsorption of water molecules. Such processes involve volumetric expansion and decrease leading to the disruption of renderings and earth blocks.

### 2.16.9.1.3 Direct inspection

Direct visual inspection and endoscopy are envisaged, on the basis of which existing wall sections can be complemented.

### 2.16.9.1.4 Deep inspection by means of NDT and MDT. Aims, methods and technologies

The use of radar technologies available in the BAM as a means to further investigate structural discontinuities will depend on calibration attempts in the BAM laboratories. In general terms, radar procedures can be employed to identify the position of large voids, crack, the inclusion of steel or wooden elements in the masonry, to qualify damage to walls, to define the presence and amount of moisture in a wall, and finally to detect a wall’s morphology (Binda and Saisi 2009).

### 2.16.9.1.5 Other inspection works

Desk Study of Site Foundations and Geology: As explained by Jaquin (2004), some of the damage which the building exhibits is most likely to be an effect of differential ground settlement. Both static and dynamic analyses will be carried out in relation to data on the foundation soil, which has an effect on the mechanical behaviour of the whole structure which cannot be neglected as it represents the restraints conditions of the structure. Data will be gathered and presented as part of the case study.

### 2.16.10 Proposed monitoring

#### 2.16.10.1.1 Static monitoring

Static monitoring by means of digital crack meters will be carried out. The envisaged type of monitoring is currently used as NDT in historical buildings diagnosis (Binda and Saisi, 2009; Bosiljkov 2010). Crack opening monitoring can be used as a complement to static and dynamic monitoring systems with the purpose to investigate the effect of humidity change/on-going damage on different monitored parameters (displacements, rotations, environmental vibration).

The use of digital crack monitoring is essential in understanding the crack pattern of the building, which in turn is essential in formulating a diagnosis for the given damage. Seasonal and daily variations can thus be assessed and results duly interpreted, when considered in conjunction to...
environmental climatic parameters (temperature, humidity, wind speed and direction) in order to allow an adequate interpretation of the results.

Crack measurements have been carried out on the Ambel complex since 1997, once or twice a year. This data, as well as the visual crack pattern data derived from the geometric survey, will be complemented by means of digital crack monitoring. This is envisaged on both the heavily damaged north east tower and, as a benchmark, on cracks which are known to be subjected to a lesser damage progress.

A possible way of applying crack openings monitoring consists of its implementation during a limited period of time (of about 1 month) at opposite moments of the annual thermal cycle (in summer and also in winter) for which the occurrence of maximum and minimum temperatures are expected (earthen buildings are particularly sensitive to these factors). Crack openings monitoring is useful to better interpret daily or seasonal variations on measures related to both static monitoring (displacements, rotations, crack openings…) and dynamic monitoring (environmental vibrations, natural frequencies…).

The crack openings monitoring system has been tested on some walls sample of earthen materials at BAM laboratory giving the possibility to monitoring the crack openings variation due to humidity factors.

2.16.10.2 Dynamic monitoring

Dynamic monitoring by means of environmental vibration measurements is envisaged as a means to measure, record and transfer velocity parameter data relating to a number of selected locations by means of geophones located at each floor and roof levels of the four-storey complex.

Results would be then used to determine the main dynamic parameters (natural frequencies, damping, modal shapes) of the Ambel complex.

These, in turn, can be used to calibrate and adjust structural numerical models (integrated modelling technology).

Dynamic monitoring is envisaged to only focus on the most damaged part of the building complex, that is to say the North-East tower and will be used as a means for both identifying modal parameters and possibly as a means to monitor the damage progression on the building by means of modal response variations.

The dynamic monitoring system is used to measure, record and also transfer the records of the velocity parameters of some selected points using geophones, as already presented in the Technology table presented by BAM. The system includes an analogical-digital converter and a data acquisition system (via PC). The dynamic set is implemented for the following purposes:

- Dynamic identification tests (dynamic test) in which velocity values are being recorded in different steps and after are being used to extract dynamic parameters (natural frequencies, mode shapes and damping) with the help of signal processing software (LABview).
- Determining the main dynamic parameters (natural frequencies, damping, modal shapes) of global structure or individual structural members

The dynamic parameters are very useful to calibrate and adjust structural numerical models (integrated modelling technology). In particular, modal updating can be performed in an iterative way by utilizing criteria like MAC (Modal Assurance Criteria) based on comparing experimental dynamic parameters with numerical dynamic parameters. In the integrated modelling technology, we make full use of the modern test technique and digital imitate technique. We apply finite element analysis software to establish preliminary model, compare it with whole structural dynamic behaviour obtained by pulsating method, then modify some restriction and parameter of the model, and fit the theoretic data with the test result to get a reasonable analysis model.

2.16.11 Proposed structural analysis. Aims and methods

2.16.11.1 Modeling and structural analysis methods
The analysis model of dynamic behaviour of historic buildings is an important basis for its vulnerability assessment. The dynamic behaviour can be obtained by establishing theoretic mechanical models.

Linear elastic FEM Modelling validated via results from dynamic monitoring will first be carried out as part of the proposed integrated modelling technology, which proceeds as follows:

- Survey the structural parameters; estimate the adjusting range, set up the preliminary analysis model (modal elastic analysis).
- Test the dynamic behaviour on site, acquire whole structural dynamic behaviour.
- Establish sensitivity system, optimize structural adjusting parameters.
- Compare testing data, simulate model parameter.

The aim of the model is to then assess the vulnerability of the structure, hypothetically to seismic events, by means of a pushover analysis. This model will be based on the previously carried out detailed geometric investigation will only cover part of the building complex at Ambel, depending on the dynamic investigation.

2.16.11.2 Model updating and monitoring. Interaction with monitoring

A preliminary Finite Element model based on results from the first on site investigation, geometry of the building and known material parameters will be constructed. When mechanical parameters of the building are not available due to difficulties in extracting historic material, results from WP4 will be used. The modal elastic analysis will contribute in the determination of where the sensors are located. After dynamic testing, a detailed numerical model based on corrected values can be used to obtained an accurate seismic vulnerability assessment of the studied building in the Ambel complex. This process is expressed in the form of a diagram below.

2.16.11.3 Assessment of possible intervention methods

Although the derived model could be a good ground on which to assess possible intervention methods by means of further models which would include the intervention methods themselves, this is not currently envisaged as part of the Case Study.
2.16.12 Envisaged interventions

2.16.12.1 Considerations on the need for conservation, repair and maintenance

The north west tower at ambel exhibits considerable cracking damage, the causes of which are believed (gerrard, 2003; jaquin, 2004) to lie in the long history of failure and repair of different parts of the building. Any intervention would need to take this aspect into account.

Main damages to the building consist of extensive cracking (6-9m cracks) between the gable wall, which is leaning outwards, and the walls it is connected to perpendicularly. The depth of these cracks is unknown, but they almost likely cause wall-wall discontinuity. Ground settlement at the north-eastern part of the site is likely to be the cause of the leaning out of the gable wall.

Additional cracking on the eastern wall is likely to be due to exceedingly high point loads which are not redistributed by wall plates or similar. Lack of a load redistributing structural element located under the floor and ceiling beams. The importance of a load redistributing element under the ceiling beams is due to the high incidence of crack formation at beam supports caused by localised high stress damage.

Anchors introduced as an attempt to arrest the progression of gable wall outward movement are indicated by worsening of cracking to be ineffective. Crack stitching conducted simultaneously has also proven to be ineffective.

Alternative measures to arrest the worsening of the building’s condition are therefore required.

Intervention methods suggested by Jaquin (2004) are crack filling and either the introduction of polypropylene bands or of grouted anchors, as opposed to the current grouted anchors, to achieve some structural continuity between gable wall and perpendicular walls. These will not be carried out within NIKER.

If the necessary concessions are obtained, the grouting material developed at BAM will be applied locally on a small scale in order to monitor durability, compatibility and adhesion to the historic material over time.

2.16.12.2 Considerations on the need for seismic strengthening

The region is only subjected to moderate earthquakes. Strengthening measures considered relate to the current state of disrepair of the building, unrelated to seismic loading.

The proposed case study finds itself in such poor conditions, that more relevant than seismic strengthening is perhaps the guarantee that feasible alternatives to the current inefficient static strengthening solutions are found. In addition, the grounds for the ineffectiveness of the current repair means should be found.

2.16.13 Potential contribution to the project

2.16.13.1 Compliance with project objectives

The Preceptory at Ambel provides an excellent opportunity to define understand damage in earthen structures and therefore to contribute to static and dynamic vulnerability reduction methodologies for earthen heritage buildings. Since over 10% of World Heritage properties are earthen architectural sites (UNESCO 2010), in 2001 Earthen architecture was approved by the World Heritage Committee as a thematic programme aimed at developing suitable methods for a sustainable conservation of earthen heritage and to ensure that best on site practices are broadly disseminated.

Nevertheless, knowledge of earthen building seismic retrofitting methodologies is fragmented, and the choice of techniques is often arbitrary and not based on coherent testing campaigns, as was already highlighted in WP3. While some of these intervention measures, such as the introduction of reinforced concrete elements, or steelwork or the superficial covering of the earthen material with water repellents such as bituminous or latex-based paints has luckily been shown not to be effective in fact rather to damage the durability or seismic performance of earthen buildings, little
dissemination of the evidence of such damage has taken and few alternatives or knowledge as to their effectiveness are available to on-site practitioners.

The facts that much earthen architectural heritage is located in developing countries where conservation principles are not necessarily followed and that retrofitting techniques are often based on increasing the overall stiffness of a structure, which is particularly harmful in earthen structures due the material’s very low stiffness are further indicators that the research on earthen buildings envisaged within the NIKER project, when suitably disseminated, would be of great benefit to their on-site repair, conservation, maintenance and seismic retrofitting practice.

Thanks to the assessment carried out in WP3 and to the testing campaigns of WP4 and WP6, the most suitable earthen vulnerability reduction and retrofitting techniques will be defined for the proposed case study.

The proposed case study aims at applying the most suitable repair techniques on the basis of an assessment of the pre-existing damages of the structure, which has up to now only been based on destructive and invasive techniques.

Opportunities offered by the Case Study are related to its current location, which is in a moderately seismic area, and to its current state of disrepair.

2.16.13.2 Foreseen opportunities for application of new approaches and technologies
The building provides an excellent opportunity to test digital crack monitoring, dynamic monitoring and integrated modelling on an earthen building. Similar studies on earth buildings are not known by the author to have been previously carried out.

2.16.13.3 Foreseen opportunities for testing and validation of proposed intervention methods
The validation of FEM modelling parameters can be carried out thanks to the dynamic monitoring data. The modal analysis of the structure and its pushover analysis therefore gain in reliability.

The proposed integrated modelling method has the advantage of being a fully respectful, non-destructive and non-obtrusive technique. Its importance lies in the lack, to the present date, of correlating structural FEM models to data deriving from monitoring of real buildings.

2.16.13.4 Possibility of monitoring long-term effectiveness of intervention
Depending on local concessions, the building might provide an opportunity to test the durability of the grouting material developed by BAM in Work Package 3 and its adhesion and compatibility with the historical earthen material at Ambel.

2.16.13.5 Other specific opportunities provided by the proposed case study

2.16.13.6 Adequacy for application to architectural heritage
The proposed case study is an important step in the development of suitable guidelines in WP 10, which contributes in ensuring that the proposed methodology is useful and relevant to on-site practice and that theoretical methods developed are feasible, as difficulties encountered on-site might not those encountered in the laboratories.
Table 2.35 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th></th>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>1- Inspection</td>
<td></td>
<td></td>
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<tr>
<td>(specify methods</td>
<td></td>
<td></td>
</tr>
<tr>
<td>and technologies)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 Geometric</td>
<td>Survey from 1957 available</td>
<td>Laser meter survey to be carried out in any</td>
</tr>
<tr>
<td>survey</td>
<td></td>
<td>case. Visual inspection and photographic</td>
</tr>
<tr>
<td>1.2 Materials</td>
<td>Construction materials are identified but</td>
<td>X-ray diffractometry, moisture tests, salinity</td>
</tr>
<tr>
<td></td>
<td>not known to have been characterised in</td>
<td>tests to assess material composition, moisture</td>
</tr>
<tr>
<td></td>
<td>physical, chemical and mechanical terms</td>
<td>and salt contents and if it is possible to</td>
</tr>
<tr>
<td>1.3 Internal</td>
<td></td>
<td>extract intact material mechanical tests</td>
</tr>
<tr>
<td>morphology</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.4 Other</td>
<td>Detailed historic evolution showing building</td>
<td></td>
</tr>
<tr>
<td></td>
<td>sequence and material sequence is available</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Gerrard, 2003)</td>
<td>None envisaged</td>
</tr>
<tr>
<td>2. Monitoring</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 Static</td>
<td>Crack monitoring once or twice a year</td>
<td>Digital crack monitoring</td>
</tr>
<tr>
<td>monitoring</td>
<td>since 1997</td>
<td></td>
</tr>
<tr>
<td>2.2. Dynamic</td>
<td>None</td>
<td>Environmental vibration measurement by means</td>
</tr>
<tr>
<td>monitoring</td>
<td></td>
<td>of geophones</td>
</tr>
<tr>
<td>3. Structural</td>
<td>Integrated FEM modelling with parameter</td>
<td></td>
</tr>
<tr>
<td>analysis</td>
<td>optimization based on dynamic monitoring;</td>
<td></td>
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<tr>
<td></td>
<td>modal analysis</td>
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<tr>
<td>3.1 Modeling and</td>
<td></td>
<td></td>
</tr>
<tr>
<td>model updating</td>
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<td></td>
</tr>
<tr>
<td>3.2 Seismic</td>
<td>Pushover proposed to assess seismic</td>
<td></td>
</tr>
<tr>
<td>analysis</td>
<td>vulnerability possible, despite the</td>
<td></td>
</tr>
<tr>
<td>4. Intervention</td>
<td>building being located in moderate seismic</td>
<td></td>
</tr>
<tr>
<td></td>
<td>seismic hazard area</td>
<td></td>
</tr>
<tr>
<td>4.1 Characterization</td>
<td>Damages identified by Jaquin (2004) are</td>
<td></td>
</tr>
<tr>
<td>of needs for repair,</td>
<td>mainly related to</td>
<td></td>
</tr>
<tr>
<td>maintenance and</td>
<td>ground settlement underneath</td>
<td></td>
</tr>
</tbody>
</table>
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

<table>
<thead>
<tr>
<th>(seismic) strengthening</th>
<th>northern tip of north east tower and excessive point loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 Design of strengthening intervention</td>
<td>-</td>
</tr>
<tr>
<td>4.2 Real implementation</td>
<td>Introduction of steel anchors in year 2000 to prevent gable wall from leaning outwards further; ineffective.</td>
</tr>
<tr>
<td>4.3 Monitoring during and after strengthening implementation</td>
<td>No strengthening currently envisaged</td>
</tr>
</tbody>
</table>

Local small scale implementation of grouting material developed in WP 3 to assess feasibility, adhesion, compatibility and durability; this is not carried out as repair and depends on permission.

2.16.14 References

2.17 BELL TOWER OF SAN VITTORE CHURCH IN ARCISATE, ITALY

2.17.1 Name, location and description

The bell Tower of the St. Vitale Church is in Arcisate, in Italy close to Varese. The investigated bell tower (Figure 2.295), about 37.0 m high, is built in stonework masonry and connected, on the East side and partly on the South side, to the XIth century church of San Vittore in Arcisate (Varese).

As shown in the pictures of Figure 2.295, the tower is characterized by irregular stonework. In the upper part, six orders of floors are present, with five of them being defined by masonry offsets at the corners and by corresponding sequences of small hanging arches marking the floor levels; the last two orders were probably added in the 18th century to host the bell trusses. The masonry texture appears often highly disordered with the local prevalence of continuous or little staggered vertical joints.

![View of the investigated tower](image1)
![South front](image2)
![East front](image3)
![North front](image4)
![West front](image5)

Figure 2.295 - (a) View of the investigated tower; (b) South, (c) East, (d) North and (e) West front.
2.17.2 Historical note

The first historic document on the tower dates back to XVI century, even probably built on a previous roman building and modified along the centuries. Six orders of floors are present; the last two orders were probably added in the 18th century to host the bell trusses.

2.17.3 Historical and artistic significance

The tower beside its clear documentary and historical importance within the contest of the Varese province represents a meaningful case-history of the several similar bell-towers in the area. Archeological inspection are ongoing, in order to detect ancient roman structures.

2.17.3.1 Structural and material features

The tower, about 37 m high, is built in stonework masonry on a square base and connected on the East side and partly on the South side to the body of the San Vittore Church (XI century).

2.17.4 Present and foreseen future use. People at risk

The present use, and the future one, is as a bell-tower of the adjacent church, which is widely frequented by the parishioners and tourists. Being within the urban center, the control of its safety has a strategic importance.

2.17.5 Considerations on valuable cultural contents

The tower does not contain precious elements. However, being very close to the church, eventual damage could involve several painting of the XVII century inside the church.

2.17.6 Present condition and damage

2.17.6.1 Considerations on the present condition

Due to erosion of the mortar joints, it is difficult to distinguish between insufficient stone interlocking and vertical cracks. The crack pattern (Figure 2.296) has been accurately surveyed also by using...
an aerial platform, which allowed to closely inspecting the external wall surface and detecting the flaw paths.

2.17.6.2 Main observed damage and decay
Along all sides, the tower exhibits long vertical cracks, most of them cutting the entire wall thickness and passing through the keystones of the arch window openings. These cracks are especially detected between the second / third order of the tower and the base of the belfry and show a maximum aperture corresponding to the upper end. Many superficial cracks are also diffused, particularly on the North (Figure 2.295 d) and West (Figure 2.295 e) fronts, which are not adjacent to the church.

2.17.7 Information on local seismicity

2.17.7.1 Local seismicity
Since 2003 all the Italian territory is considered seismic area.

2.17.7.2 Characterization of the seismic action
More specifically the town of Arcisate is in the lower seismic category (PGA 0.05-0.07g)

2.17.8 Previous and on-going studies

2.17.8.1 Inspection works
Few tests were carried out previously, due to the budget problem of the Parish. However the testing campaign carried out could be considered an example of minimal program aimed at the control of the structural safety.

2.17.8.1.1 Geometric survey
A prompt geometric survey was carried out aimed at the tilting control and the structural analysis. The masonry unregularity and the surrounding building do not allow the use of refined survey tools.

2.17.8.1.2 Research on materials
The masonry has been also characterized through sonic tests and the results of sonic velocity generally indicate a relatively compact masonry, of fairly good execution.

2.17.8.1.3 Visual inspection
The crack pattern has been accurately surveyed also by using an aerial platform, which allowed to closely inspecting the external wall surface and detecting the flaw paths.

2.17.8.1.4 Other inspection works
Ambient vibration tests. Two ambient vibration tests were conducted on the tower, on June 2007 and June 2008. In both tests, a 16-channel data acquisition system with 15 uniaxial WR 731A piezoelectric accelerometers were used. For each test, two different series of ambient vibration data were recorded: in the first series, referred in the following as AV1, the ambient excitation was only provided by the wind and the micro-tremors; in the second series, referred in the following as AV2, the excitation was provided by the swinging of bells. In both series, the well-known rule of thumb about the length of the time windows acquired (that should be at least 1000 times the period of the structure’s fundamental mode) was largely satisfied. The sample rate was 200 Hz to provide good waveform definition. An example of the acceleration time-histories recorded in June 2007 test in the upper part of the tower is given in Figure 2.17.3(a)-(b). An important remark concerns the significant increase of the vibration level associated to ambient excitations compared to micro-tremors, as it is shown in Figure 2.17.3 (a)-(b); the maximum amplitude of acceleration responses is increased of about 20 times by the bell swinging. Similar results have been obtained in the test performed on June 2008.
Mode identification The extraction of modal parameters from ambient vibration data was carried out by using two different output-only techniques: the Frequency Domain Decomposition (FDD, Brincker et al. 2000) in the frequency domain and the data driven Stochastic Subspace Identification (SSI, van Overschee and De Moor 1996) in the time domain; these techniques are available in the commercial program ARTeMIS (SVS 2010). In order to compare the mode shapes identified using different methods and different test data, the Modal Assurance Criterion (MAC, Allemang and Brown 1983) was computed.

Experimental evidence 5 vibration modes were identified in the frequency range of 0-6 Hz by FDD and SSI techniques applied to AV1 data series. The results of OMA in terms of natural frequencies can be summarized through the plots of Figure 2.17.4 (a)-(b); the figures show the lower Singular Values (SV) of the spectral matrix (FDD, Figure 2.17.4 (a)) and the stabilization diagrams (SSI, Figure 2.17.4 (b)), respectively. Figure 2.17.4 (a) highlights the effectiveness of FDD technique in the mode identification through well-defined local maxima in the 1st SV line; similarly, Figure 2.17.4 (b) shows the alignments of the stable poles in the stabilization diagram of the SSI method, identifying the vibration modes, as well. Furthermore, Figure 2.17.3(a)-(b) clearly show the correspondence of the natural frequency estimates between the two techniques, with the resonant peaks of Figure 2.17.4 (a) being placed at the same frequencies of the alignments of stable poles of Figure 2.17.4 (b).

Figure 2.17.4 (c) shows the identified mode shapes (June 2008, FDD technique): dominant bending (B) modes were identified at 1.21 (B1), 1.29 (B2), 3.98 (B3) and 4.14 Hz (B4) while only one torsion mode (T1) was identified at 3.56 Hz. It is observed that the dominant bending modes of the tower involve flexure along the diagonals of the tower.

Further remarks derive from the comparison of the modal parameters identified from the two different data-sets acquired, under different level of ambient excitation, on June 2007, (Table 2.17.1): the comparison reveals slight but systematic decreases of the frequencies associated to the higher level of excitation caused by the bell swinging (Table 2.17.1). Furthermore, significant differences are detected between the mode shapes identified from data series AV1 and AV2. Similar results were obtained by applying the SSI technique and in the test performed on June 2008, even if with less marked differences in the frequency shift. The comparison of the mode shapes identified in the two series of tests clearly highlights that the MAC in the 2007 test tends to decrease as the order of mode increases; for the two upper modes the MAC is between 0.87 and 0.80. In the June 2008 test, the MAC seems more stable.

Hence, the dynamic characteristics of the tower are possibly dependent on the amplitude of excitation/response.
Table 2.36 - Correlation between the modal parameters identified in 2007 and 2008.

<table>
<thead>
<tr>
<th>Mode Type</th>
<th>$f_{AV1}$ (Hz)</th>
<th>$f_{AV2}$ (Hz)</th>
<th>$\Delta f / f$ (%)</th>
<th>MAC</th>
<th>$f_{AV1}$ (Hz)</th>
<th>$f_{AV2}$ (Hz)</th>
<th>$\Delta f / f$ (%)</th>
<th>MAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>B&lt;sub&gt;1&lt;/sub&gt;</td>
<td>1.211</td>
<td>1.191</td>
<td>-1.65</td>
<td>0.997</td>
<td>1.211</td>
<td>1.201</td>
<td>-0.83</td>
<td>0.982</td>
</tr>
<tr>
<td>B&lt;sub&gt;2&lt;/sub&gt;</td>
<td>1.289</td>
<td>1.260</td>
<td>-2.25</td>
<td>0.986</td>
<td>1.270</td>
<td>1.260</td>
<td>-0.79</td>
<td>0.955</td>
</tr>
<tr>
<td>T&lt;sub&gt;1&lt;/sub&gt;</td>
<td>3.564</td>
<td>3.525</td>
<td>-2.25</td>
<td>0.870</td>
<td>3.525</td>
<td>3.467</td>
<td>-1.65</td>
<td>0.942</td>
</tr>
<tr>
<td>B&lt;sub&gt;3&lt;/sub&gt;</td>
<td>3.984</td>
<td>3.877</td>
<td>-2.69</td>
<td>0.804</td>
<td>3.984</td>
<td>3.906</td>
<td>-1.96</td>
<td>0.936</td>
</tr>
<tr>
<td>B&lt;sub&gt;4&lt;/sub&gt;</td>
<td>4.141</td>
<td>4.059</td>
<td>-1.98</td>
<td>0.804</td>
<td>4.141</td>
<td>4.063</td>
<td>-1.88</td>
<td>0.982</td>
</tr>
</tbody>
</table>

Figure 2.297 - June 2008, data series AV1: (a) Singular Values of the spectral matrix and selected modes (FDD technique); (b) Stabilization diagram (SSI technique); (c) Vibration modes (FDD).

2.17.8.2 Monitoring

A static monitoring was installed aimed at the monitoring of the crack opening.

2.17.8.3 Structural analysis

F.E. Modelling and Model Tuning The experimental investigation was preceded by the development of a 3D finite element model (Figure 2.17.5), based on the available geometric survey. The F.E. program SAP2000 was used to create the numerical model. The tower was modelled by using 8-node brick elements. A relatively large number of finite elements have been used in the model, so that a regular distribution of masses could be obtained and all the openings in the load-bearing walls could be reasonably represented. The model consists of 3481 solid elements with 17196 active degrees of freedom.

Since the geometry of the tower was accurately surveyed, the main uncertainties are related to the boundary conditions and the characteristics of the material. In order to reduce the number of uncertainties in the model calibration, the following assumptions were introduced: (a) the weight per unit volume of the masonry was assumed as 17.0 kN/m$^3$; (b) the Poisson’s ratio of the masonry was held constant and equal to 0.15; (c) since the soil-structure interaction is hardly involved at the low level of ambient vibrations that existed during the tests, the tower footing was considered as fixed.

Subsequently, a sensitivity analysis was carried out in order to explore the influence of the average elastic characteristics of stone masonry and of the connection (Figure 2.295) between the tower and the neighbouring building on the available dynamic characteristics (natural frequencies and mode shapes).

Once the sensitivity analysis provided a confirmation on the correct choice of the updating parameters, the FE model was refined in 3 steps of systematic manual tuning, varying within each
step the assumption of the material from isotropic (FEM1) to orthotropic (FEM2), and considering the contribution of the adjacent buildings by a spring series (FEM3). In the last step (FEM4), the uncertain structural parameters were identified in order to enhance the correlation between experimental and numerical modal behaviour using the technique described in (Douglas and Reid 1982).

In the manual tuning, the correlation between the dynamic characteristics of the FE models and the experimental results was evaluated via the maximum absolute frequency discrepancy $D_{F,max}$:

$$D_{F,max} = \max(D_{F,i})$$

(1)

$$D_{F,i} = 100 \frac{|f_{FEM,i} - f_{FDD,i}|}{f_{FDD,i}}$$

(2)

and the average frequency discrepancy $J$ (Table 2.36):

$$J = \frac{1}{M} \sum_{i=1}^{M} D_{F,i}$$

(3)

**FEM1** A preliminary dynamic analysis was performed to check the similarity between experimental and theoretical modal parameters. In this analysis, the Young’s modulus of stone masonry was assumed equal to 3.00 GPa. It is worth noting that the assumed value was suggested either by engineering judgement or by the results of sonic tests while exhaustive tests to evaluate the mechanical characteristics of the stone masonry are not yet available.

The comparison between theoretical and experimental modal parameters shows highly imperfect correlation, since the average frequency discrepancy ranges up to about 21.63% with the maximum value of 57.99% (Table 2). Beyond that, FEM1 does not accurately represent the structural behaviour of the tower since:

- the model is much stiffer than the tower (with all the natural frequencies of FEM1 model significantly exceeding the experimental ones);
- the torsion mode $T_1$ of the model does not follow the experimental sequence, where the torsion mode is placed between two couples of bending modes; on the contrary, the torsion mode follows two couples of bending modes in FEM1 model;
- also the mode shapes of bending modes exhibit major differences with the experimental results. Specifically, FEM1 bending modes involve motion along the main N-S and E-W directions while the identified tower modes involve bending along the diagonals (Fig. 4(c)).

**FEM2** The poor quality of correlation clearly indicates that the assumptions on the isotropic behaviour of stone masonry and on the connection with the neighbouring building need to be revised. Hence, an orthotropic elastic behaviour (FEM2) was assumed for the stone masonry; the average characteristics of the material were $E = 3.00$ GPa, $G_{13} = G_{23} = 0.33$ GPa. The introduction
of orthotropic elasticity dramatically improved the correlation with the experimental results (Table 2). In particular:

the stiffness of the model significantly decreased, so that the average and maximum frequency discrepancies are reduced from 21.63% to 8.52% and from 57.99% to 17.11%, respectively;

the torsion mode \( T_1 \) of the FEM2 model correctly follows the experimental sequence.

On the other hand, the flexural modes of FEM2 continue to exhibit bending along the main N-S and E-W directions, differently from the observed modes (involving bending along the diagonals).

FEM3 A third model FEM3 was subsequently developed, by accounting for the connection between the tower and the church through a series of linear (nodal) springs of constant \( k \). After some manual tuning, the stiffness of springs \( k = 4 \times 10^4 \) kN/m was assumed. As it had to be expected from previous investigations, now the bending modes are fully consistent with the experimental results; in addition, further reduction of frequency discrepancies was attained (Table 2), with \( J \), the average frequency discrepancy less than 5.91%. Of course, the value of \( D_{F,max} = 11.29\% \) (obtained for mode \( T_1 \)) remains quite high and the differences are surely to be related to the simplified distribution of the model elastic properties, which were held constant for the whole structure. However, the correlation between theoretical and experimental behaviour is satisfactory and provides a robust verification of the FEM3 model main assumptions, being a one-to-one correspondence between the mode shapes.

FEM4 The uncertain structural parameters \((E, G_{13}=G_{23}, k)\) were finally estimated by minimizing the difference between theoretical and experimental natural frequencies through the procedure proposed by Douglas and Reid (1982). The optimizing iterative procedure identified more accurately the uncertain values concerning the material properties and the spring stiffness used in the updated model, as follows: \( E=2.64 \) GPa, \( G_{13}=G_{23}=0.37 \) GPa, \( k=3.33 \times 10^4 \) kN/m. Once the optimal estimate of the parameters of FEM4 model was performed, a complete correlation analysis between the theoretical and experimental modal parameters was carried out (Figure 2.17.6). The results shows a very good agreement of the model with the experimental results, being \( D_{F,max} \) equal to 7.56% and particularly low for the higher bending modes, in spite of the simplified distribution of the elastic properties. Hence, the model seems suitable for a successive investigation, including further non-destructive tests on the materials and the application of more refined system identification techniques.

| Table 2.37 - Correlation between the experimental results and the computed frequencies |
|----------------------------------------|--------|--------|--------|--------|--------|--------|--------|
| \( B_1 \) | \( B_2 \) | \( T_1 \) | \( B_3 \) | \( B_4 \) | \( J \) | \( D_{F,max} \) |
| FDD | 1.211 | 1.270 | 3.525 | 3.984 | 4.141 | – | – |
| FEM1 | 1.220 | 1.246 | 5.569 | 4.982 | 5.072 | 21.63 | 57.99 |
| FEM2 | 1.123 | 1.141 | 2.922 | 3.876 | 3.919 | 8.52 | 17.11 |
| FEM3 | 1.183 | 1.141 | 3.127 | 4.126 | 4.233 | 5.91 | 11.26 |
| FEM4 | 1.135 | 1.174 | 3.265 | 4.091 | 4.174 | 4.94 | 7.56 |
2.17.9 Proposed monitoring

2.17.9.1 Dynamic monitoring

In order to better understand and explore the peculiar behaviour found by dynamic testing a continuing dynamic monitoring has been recently installed.

On the base of the dynamic testing evidence, 3 accelerometers Dytran 3191A1 (10V/g) were placed on the structure continuing acquiring at 200Hz the acceleration.

The data processing is ongoing, as well as the calibrating of a prompt alarm procedure and a control software based on the comparison of modal parameters.

Due to its characteristics, the installed monitoring system is suitable for the monitor the present situation but also the intervention and post intervention phases.
The research points out the main steps of a prompt procedure based on experimental data from dynamic testing for the general structural evaluation and model calibration. The model will be developed step by step, including experimental evidences and refining the analytical assessment procedures.

Radar interferometry, an interesting new technique will be tested on a historic building.

2.17.10.3 Potential contribution to validate the project's methods and criteria

Invasive intervention without a deep knowledge of the building could be potentially dangerous. The availability of a reliable calibrated monitoring procedure able to promptly detect alarming changes of the behaviour is a precious opportunity, giving time to carry out further refining tests or addressed design.

2.17.10.4 Possibility of monitoring long-term effectiveness of intervention

The monitoring is suitable to work for years, due to the simplicity of the system, which measure the structural behavior in 3 points. However after the intervention the dynamic tests should be repeated in order to update the reference modal parameters.

Table 2.38 - Summary of past, on-going and envisaged new activities.

<table>
<thead>
<tr>
<th>2. Monitoring</th>
<th>Already developed or on-going activities</th>
<th>Activities to be developed within the project</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 Static monitoring</td>
<td>On going</td>
<td>Data processing</td>
</tr>
<tr>
<td>2.2 Dynamic monitoring</td>
<td>Recently installed</td>
<td>Calibrating alarm procedure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Radar interferometry application</td>
</tr>
<tr>
<td>3. Structural analysis</td>
<td></td>
<td>Model updating</td>
</tr>
<tr>
<td>3.1 Modeling and model updating</td>
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</table>