

**The First International
Workshop on
Restoration and
Strengthening of
Historical Structures**
Volume I: Papers

Iranian Ministry of Cultural Heritage, Tourism and
Handicrafts (MCTH)
and
International Scientific Committee on the Analysis
and Restoration of Structures of Architectural
Heritage (ISCARSAH)

The First International Workshop on Restoration and Strengthening of Historical Structures

Ed. María Margarita Segarra Lagunes, Mehrdad Hejazi

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The First International Workshop on Restoration and Strengthening of Historical Structures

Ali Darabi

Deputy Head of Iran's Ministry of Cultural Heritage,
Tourism and Handicrafts

Ever increasing growth and rapid changes of science and technology in today's world and the rise of emerging challenges makes education and innovation necessary in all areas.

Every scientific and dynamic society must make use of different knowledge and experiences to progress, thus with emphasizing the necessity of using domestic experts and specialists to protect and restore the historical and cultural monuments of Iran, which is in line with the current policies of resistance economy, it is also necessary to prioritize the use of international experience in order to equip the domestic experts with required knowledge.

This effort which has been carried out by the Office of Conservation and Rehabilitation of Historic Monuments, Sites and Urban Fabrics in collaboration with the specialists of the International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH), serves as a starting point for gathering information, training and familiarizing experts with the world's best practices for protecting and Strengthening historical buildings.

However, besides the probable problems and shortcomings which exist in various strengthening methods, the above workshop serves as a starting point for the massive collec-

tion of knowledge on consolidation of historic monuments, improving the performance, and providing valuable and up-to-date experiences to experts and operators of the restoration and rehabilitation of valuable monuments of Iran And is an innovative step in the indigenization of international technology and innovations in the country.

One of the strengths of this workshop is the comprehensive collection of important and influential international ideas in the field of restoration of architectural heritage structures in recent decades, which, while transferring the knowledge and experience of prominent experts, is an important step in reducing the mistakes in restoration and strengthening of historical monuments.

The Deputy for Cultural Heritage of Iran has the duty to familiarize its experts and stakeholders with the updated sciences and experiences of the world, therefore, holding this three-day workshop with the invitation of prominent professors of the International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) helps answering the scientific concerns of colleagues, while making an effort to share the successful experiences of Iran in the field strengthening the historical monuments with the above mentioned professionals.



Introduction

E. Görün Arun

Past-President of ISCARSAH

Paulo B. Lourenço

President of ISCARSAH

The International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) was founded by ICOMOS (International Council of Monuments and Sites) in 1996 as a forum and network for engineers, architects, and building scientists involved in the restoration and care of built heritage. Members of the ISCARSAH are composed of internationally renowned engineers, scientists, architects, specialists, and educators with representation from Europe, Asia, Australia, North and South America.

ISCARSAH committee is concerned with encouraging the structural and technical aspect of conservation, protection, rehabilitation and enhancement of structures of architectural heritage on the international level. The Committee has authored the ICOMOS Charter - Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage (ISCARSAH Principles), which was ratified by the ICOMOS 14th General Assembly in Victoria Falls, Zimbabwe in October of 2003. The Committee also drafted the ISCARSAH Guidelines, with a revised version now being proposed as an ICOMOS doctrinal text.

The objective of ISCARSAH is to:

- *Promote international co-operation in integrating the*

contribution of engineering into conservation knowledge,

- *Gather, study and disseminate knowledge and information concerning principles, techniques and policies,*
- *Participate in the organization of world-wide training programs to promote greater understanding in the structural restoration field.*

In line with ISCARSAH objectives Vice President Professor Mehrdad Hejazi proposed to organise a workshop to work on the rich heritage of Iran. In 2018 October, MCTH (Iran's Ministry of Cultural Heritage, Tourism and Handicrafts) and ISCARSAH organized an International Workshop on "Restoration and Strengthening of Historical Structures". The workshop included a three-day Training Program on "Restoration of Historical Masonry Buildings" and seven days of site visits. During these site visits, thirty-one heritage places in Tehran, Semnan, Dezful, Shoush, Shoushtar, Shiraz, Izadk-hast, Qazvin, Zanjan, Soltanieh, Hamedan, Khorramabad, Ardebil and Isfahan were visited by two groups. Each visited place was guided by experienced Iranian engineers who provided background information. At the end of the workshop, participants wrote reports for each visited heritage place, according to a prepared template, which are now gathered in this publication. At the end of the workshop, on

October 18th, the Tehran Document was also prepared and is included in this publication.

The site visits gave participating ISCARSAH members the opportunity to learn about historical sites in Iran. It is hoped that the lectures given by the ISCARSAH experts and discussions on site visits have contributed to exchange

of knowledge on structural restoration of architectural heritage and preservation of our past.

This book contains abstracts of the training seminars and reports on the visited places. It is hoped that this contents are of assistance for those who are working in the important field of heritage structures.

Representatives of the ISCARSAH Committee who reunited in Tehran on 20 to 22 October 2018 express their satisfaction for the results obtained during the discussions in the International Workshop on Restoration and Strengthening of Historical Structures (Tehran Workshop). The Committee would like to thank the Iranian government authorities, specifically the Iranian Cultural Heritage, Handicrafts & Tourism Organization for their warm and friendly hospitality.

Historical monuments and buildings with their unique and sometimes archaic use of building materials, systems and traditional building techniques present distinctive challenges. These challenges vary greatly from the challenges posed by contemporary buildings. Historical buildings can rarely be addressed using standard procedures of evaluation and structural verification that are defined in building codes that have been developed for new construction. Therefore, historical monuments and buildings set their own “standards” and the “standards” are unique to each individual monument or building.

The ISCARSAH committee has been reflecting on this subject since 1997. These reflections are a synthesis of expert thought, experience and wisdom of members from all over the world. The committee has identified similarities and differences relative to building type, building materials, construction techniques, maintenance and the cultural aspects regarding built heritage. The Committee has developed the ISCARSAH Principles and is further developing the ISCARSAH Guidelines to address this challenge.

A toolbox has been presented in the meeting for the comprehensive evaluation and potential repair and strengthening of historical monuments and buildings at the Tehran workshop. However, the development of multidisciplinary teams, the use of specific tools, and the decision for intervention will vary from building to building. This variance is caused not only by the materials, systems and building techniques that were used in the original creation and historical modifications to the building, but by the cultural environment, climate, present use, owners and stakeholders, and other factors.

We are aware of new materials and processes which show promise with the strengthening of contemporary buildings as shown by experimentation. We urge heritage conservation engineers to approach these new materials and processes with a healthy skepticism until their long-term efficiency in terms of mechanical behavior, compatibility and durability can be proven by experience in the field. Built cultural heritage should not serve as test cases for these new materials and processes.

Sustainability has become an important aspect in our era. We know that the most sustainable building is one that is already constructed and that includes all historical buildings. Adapting the use of historical buildings for other uses than those for which they were originally intended can be a strategy for saving cultural heritage. However, the use must be adapted to the monument

and not the monument to the use, to avoid detrimental impact on heritage values.

The ISCARSAH Committee is pleased and honored to offer its expertise and knowledge to the global community of cultural heritage. All engineers and architects have a responsibility for health, safety and welfare to the public. Heritage conservation professionals also have a responsibility to safeguard the heritage value of the monument. Deep knowledge of the history; art and architecture; and structural, geotechnical, environmental and technological aspects of the monument are essential before implementing any repair or strengthening strategies. With the lack of knowledge, there is always a danger that the heritage value would be detrimentally affected by unnecessary interventions. Performance criteria should always be considered to achieve a correct balance between safety and safeguarding built cultural heritage.

Papers

1. Existing Buildings and Sustainability

1.1. The history, materials, and assemblies of heritage structures present unique challenges in their evaluation, conservation, rehabilitation, restoration, and upkeep. For the conservation engineers involved in their care, there is value in the integrity of the hidden components such as structural systems as they represent increasingly rare examples of the building technology of their time.

1.2. The most sustainable building is one that is already built. Buildings represent “embodied carbon” meaning that energy has already been produced by already spent fossil fuels to harvest, quarry, manufacture, and transport construction materials; and then to erect the building. Keeping and using existing buildings avoids the release of massive greenhouse gases, and emissions caused by needless demolition and replacement of existing buildings. Retrofitting existing buildings to meet better performance standards is an effective strategy for reducing near and midterm carbon emissions, the most important step in limiting climate disruption. Existing buildings, therefore, are a resource for tackling climate change.

2. Modern Building Codes

2.1. Modern building codes are prescriptive in nature and can be understood as “one-size-fits-all.” Modern building codes often dictate methods that are often inappropriate for existing and archaic structures. Important decisions on retrofit techniques can be removed from the conservation engineer and placed in the hands of the building official. This is a situation that will render unsatisfactory results and should be avoided.

2.2. Various structural components and systems in older buildings may not meet current structural code requirements though they have been in service for centuries. But does the failure to meet current standards automatically designate a portion of a building as unsafe? A lack of engineering judgment about code regulations relative to the existing structures can cause unnecessary structural upgrades to existing buildings because the original structure does not work “on paper.” Current regulatory approaches relying extensively on the judgment of local building officials lead to unpredictable and inconsistent results, unwarranted loss of historical fabric, and excessive expense for arbitrary structural retrofits.

2.3. The main purpose of building codes is to protect *public health, safety and general welfare* as they relate to the construction and occupancy of buildings and structures. The purpose of the conservation engineer must be more focused on health, safety and welfare as well; but also, the development of schemes that entail the *minimum of intervention* and the *maximum of protection*. Such a focus requires passion to conserve the built heritage, creativity, and possibly an element of risk.

2.4. The recently ratified *ISO 13822:2010: Bases for design of structures - Assessment of existing structures* provides general requirements and procedures for the assessment of existing structures based on the principles of structural reliability and consequences of failure (Figure 1). Most importantly it stipulates that the assessment of a heritage structure will include two aspects: its structural performance; and its *value as a cultural resource*. These two aspects should be considered in any

INTERNATIONAL STANDARD		ISO 13822
Second edition (2010)		
Bases for design of structures — Assessment of existing structures		
Bases for design of structures — Evaluation des structures existantes		
Contents		Page
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Figure 1. ISO 13822, the international standard that addresses the assessment of existing structures.

decision involving possible structural interventions and should therefore be carried out in tandem.

3. Review of the ICOMOS Charters

Do international codes and standards prepared by the committees of ICOMOS guide our deliberations regarding heritage conservation and sustainability?

3.1. A review of the *International Charter for the Conservation*

and Restoration of Monuments and Sites (the Venice Charter, 1964) (Figure 2) provides the following insights:

3.1.1. Article 6: The conservation of a monument implies preserving a setting which is not out of scale. Wherever the traditional setting exists, it must be kept. No new construction, demolition or modification which would alter the relations of mass and color must be allowed.

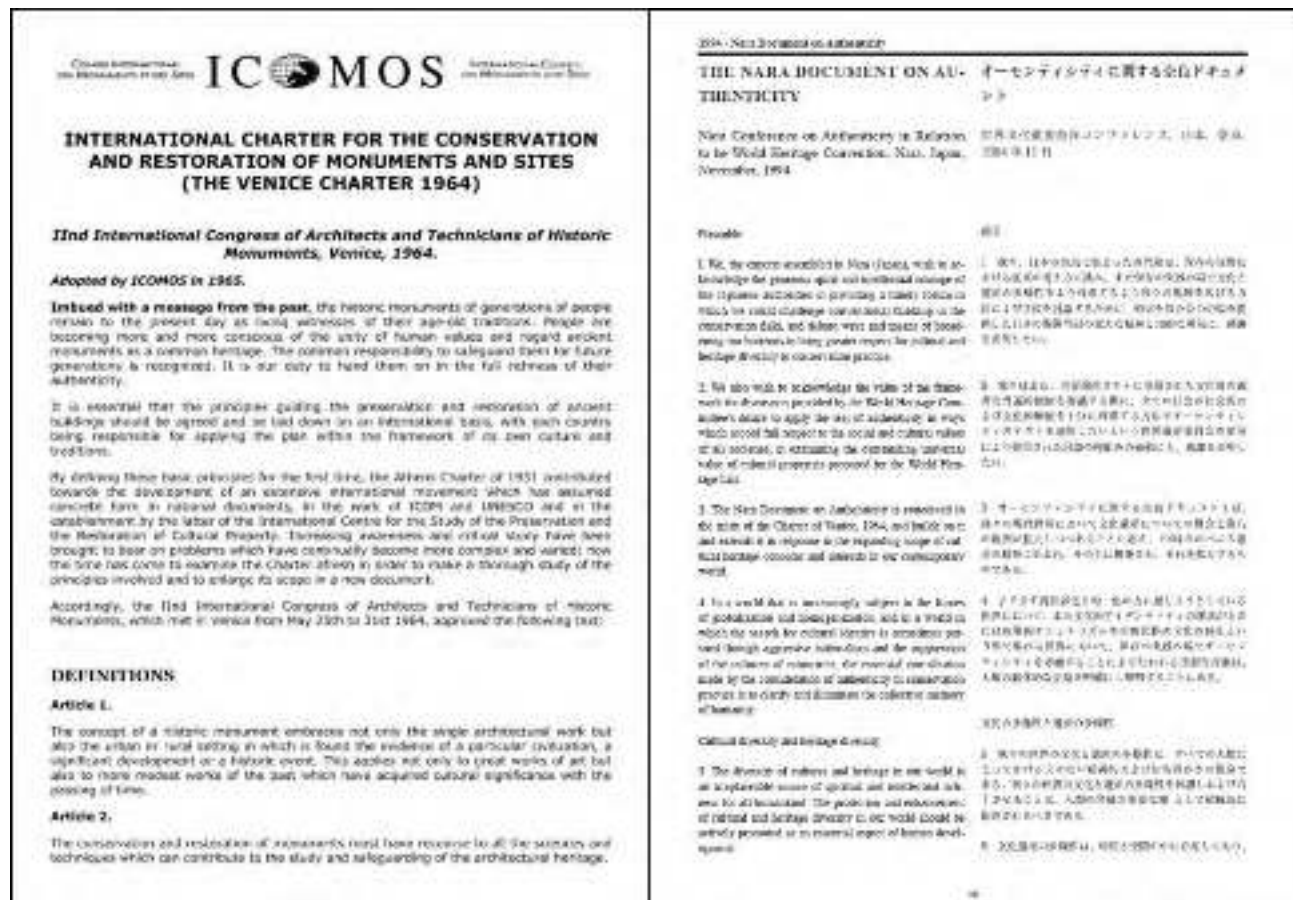


Figure 2. The Venice Charter (1964) and The Nara Document on Authenticity (1994).

3.1.2. Article 15: All reconstruction work should however be ruled out “a priori”. Only anastylosis, that is to say, the reassembling of existing but dismembered parts can be permitted. The material used for integration should always be recognizable and its use should be the least that will ensure the conservation of a monument and the reinstatement of its form.

3.2. The Nara Document on Authenticity (1994) (Figure 2) provides further guidance:

3.2.1. Article 4: In a world that is increasingly subject to the forces of globalization and homogenization, and in a world in which the search for cultural identity is sometimes pursued through aggressive nationalism and the suppression of the cultures of minorities, the essential contribution made by the consideration of authenticity in conservation practice is to clarify and illuminate the collective memory of humanity.

3.2.2. Article 6: Cultural heritage diversity exists in time and space and demands respect for other cultures and all aspects of their belief systems. In cases where cultural values appear to be in conflict, respect for cultural diversity demands acknowledgment of the legitimacy of the cultural values of all parties.

3.2.3. Article 7: All cultures and societies are rooted in the forms and means of tangible and intangible expression which constitute their heritage, and these should be respected.

3.3. The Eurocentric axioms expounded by the *Venice Charter*, and *Nara Document* informing us to respect

cultural aspects seem almost quaint today when once again armed conflict identifies built heritage and antiquities as objects to be destroyed rather than conserved. Built heritage destruction calls for reconstruction.

4. ISCARSAH Principles

4.1. Much of the guidance provided in the ISO document is in harmony with the *ICOMOS Charter: Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage (ISCARSAH Principles)* (Figure 3) which was ratified in 2003. The ISCARSAH Principles were authored by members of the International Scientific Committee of the Analysis and Restoration of Structures on Architectural Heritage (ISCARSAH). Members of ISCARSAH are composed of internationally renowned conservation engineers, heritage architects, scientists, specialists, and educators with representation from all continents.

4.2. The principles of research and documentation, authenticity and integrity, compatibility (both visual and physical), minimal intervention, and reversibility are in harmony with those that are the foundation of the Venice Charter (1964) and other charters and doctrinal texts developed for use in the heritage conservation community. Although the concepts presented are not new, their comprehensive development within the *ISCARSAH Principles* is about as poetic as one can become in a highly technical arena.

4.3. A definition of conservation engineering has been discussed countless times within the heritage conservation community, and the underpinnings of the profession are usually a short sentence or concise motto. The



Figure 3. The ISCARSAH Principles (2003) had been translated into Arabic, Bosnian, English, Farsi, French, Italian, Spanish, and Turkish.

various answers are typically in harmony with each other in that they refer to how to avoid interventions, if possible, how to limit interventions when they are necessary, and how to put to use what remains of the original structural system.

4.4. Can one consider the highly precise profession of structural engineering, relate it to the specific area of heritage conservation and then define a philosophy in just a few words? Most would agree that a decision-making philosophy would have to be further defined because of the uniqueness of heritage structures: they vary in size, shape, age, usage, material composition, structural system, geographical terrain, climatic location, the

cultures that created them and the cultures into which their care has fallen, just to name a few categories.

4.5. Conservation, restoration, and reinforcement of heritage buildings require multi-disciplinary teams of experts with the conservation engineer being only one. Engineers, however, cannot afford to turn a blind eye to all else but the detail at hand. Their recommendations, if implemented, can impact other parts of the building that are seemingly unrelated to their work. There is, therefore, a responsibility to consider a project holistically.

4.6. A medical analogy is a way to further define a philosophy. If engineers who work on existing structures



Case History: The known history of a building including past traumas, interventions, modifications, etc.

Diagnosis: Identifying or determining the nature and causes of deterioration and decay, and the opinion derived from such an investigation.

Therapy: The remedial measures (conservation, preservation, rehabilitation, etc.) in response to the diagnosis.

Monitoring: verifying and regulating the efficiency of an enacted therapy through monitoring and cyclical examination.

Figure 4. The medical analogy, cyclical rather than linear in nature, that is used for the basis of approach for the diagnosis and therapy for structures of architectural heritage.

can see themselves as “building doctors” and the buildings are “patients,” then how is their relationship to their patients the same as that of the medical doctor and patient? How is it different? And how should the similarities and differences affect the approach?

4.6.1. Buildings are alive and are living documents of our built heritage. We live in and around buildings, and they are a part of our consciousness.

4.6.2. Buildings can speak but not verbally. Experts are trained to listen to them in different ways using different technical languages.

4.6.3. Heritage buildings have a greater lifespan than we do. Here is an important distinction between the building and human doctor. Heritage buildings have a far greater lifespan than any single individual. If one compares the life of a building to the length of time of

human interventions, then these interventions become very short relative to the building’s age.

4.7. With the medical analogy one can have a philosophical framework like that used in medicine: *case history; diagnosis; therapy; and control*. These four steps listed follow sequentially (Figure 4).

4.7.1. The *case history* of a building, including past traumas, interventions, modifications, etc. is the first step, archival in nature, in any heritage conservation enterprise.

4.7.2. *Diagnosis* is the process of identifying or determining the nature and cause of damage and decay through archival research, observation, analysis, testing and the conclusions derived from these activities.

4.7.3. *Therapy* is the choice of intervention in response to the diagnosis and can include consolidation,



Figure 5. Interior wall painting at the Church of the Resurrection at Sucevita Monastery, Romania (1584-1601) tells the story of the dedication of the church.

emergency measures, conservation, restoration, rehabilitation, and strengthening.

4.7.4. Monitoring is the standard of comparison that is established for checking the results and long-term impact of the therapy.

4.8. Diagnosis: The conservation engineer is introduced to a building and its complaints and is shown its past medical records. Then an investigation to support the diagnostic phase commences that may include the following steps that are listed from basic to sophisticated:

4.8.1. Review of archival documents (Figure 5).

4.8.2. Geometric documentation of the building (Figure 6).

4.8.3. Minimally intrusive survey that includes visual examination, sounding, and probing with minimally



Figure 6. Section through the wooden Tserkva of the Descent of the Holy Spirit in Potelych, Ukraine (1502) taken from a 3D laser scan (scan by Skeiron in Lviv).

intrusive devices (Figure 7).

4.8.4. Invasive survey, i.e. removing components and making inspection openings, is likened to exploratory surgery and may be performed to reveal the nature and condition building systems and components (Figure 8).

4.8.5. Samples may be removed for laboratory testing and analysis to understand the nature, composition and condition of materials (Figure 9).

4.8.6. Computational analysis might then be performed once the nature and condition of building materials and systems are better defined (Figure 10).

4.8.7. Long-term monitoring might then be implemented to understand environmental and climatic conditions and how they relate to the structure; or to study how ambient vibrations migrate through the structure (Figure 11).



Figure 7. Prof. Giacomo Boni, famed Italian archaeologist, inspects a crack in the concrete vaulting of the dome of the Pantheon during restoration in 1925. Figure 8. Sondage at the Tangier American Legation Museum Pavilion Arabe in Morocco (early 20th Century) to determine the makeup of the foundations. Figure 9. Removal of cores through the marble floor of the Oklahoma State Capitol (1914-1918) to determine the depth of the marble staining.

4.9. The medical analogy provides a philosophical framework, but it does not provide an approach to begin the evaluation of heritage buildings. Each of the approaches presented have advantages and disadvantages. Thus, conservation engineers should and often do rely on more than one approach. A review of these approaches is presented below.

4.9.1. Intuitive Approach: Knowledge of what has occurred in the past can help to predict what might occur in the future and may provide an indication of future safety expectations. Consequently, satisfactory past performance becomes an important consideration when predicting the future service life of the building. There is a common argument that “this building has stood for a hundred years already, so it should be ready for another

hundred.” Susceptibility to such arguments is tempting because they are often presented by well-meaning heritage conservation advocates. However, such arguments, though good to construct an initial hypothesis, should not be accepted outright without thorough scrutiny as they are not scientific. Past safety is not a reliable guide to future safety when one takes the results of long-term damage and decay into consideration.

4.9.2. Inductive Approach: Inductive reasoning (bottom-up logic) consists of inferring general principles or rules from specific facts. However, inductive reasoning only states that, given the premises, the conclusion is probable. Analogous comparison of general premises can often be made between the expected behavior of a building and that of other structures that have related typologies and

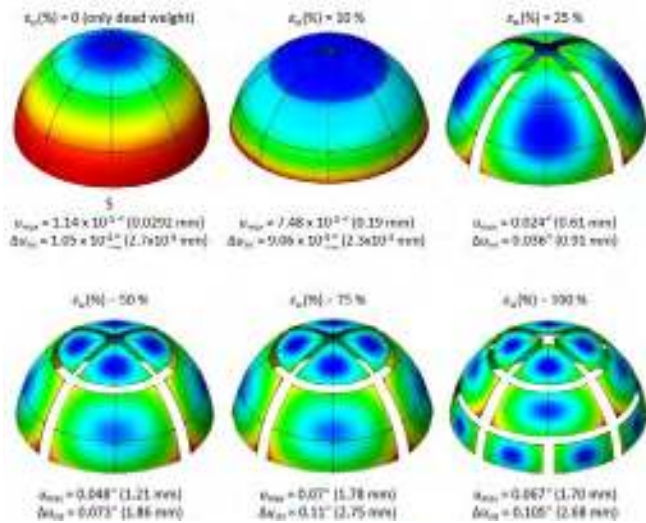


Figure 10. Concrete shrinkage studies of the nonstructural tile overlay on the dome of the Saints Constantine and Helen Greek Orthodox Church in San Diego (12979) using EEM nonlinear analysis (Midas FX+ for DIANA).

whose behaviors have already been studied and understood. The author took part in a study that examined trussed roof systems of the church typology in Chicago during an era (1854-1924) when American building was moving from empirical to engineered truss systems and from the use of wood to iron and then steel. Chicago was devastated by a fire in 1871 and this crisis afforded the opportunity for experimentation in engineering and construction. From this study generalizations were developed, a few which are listed here: metal trusses will generally last longer than trusses using wooden members; trusses with wood members are more prone to sudden collapse and fire; scissors trusses tend to deform outward by forces that are analogous to masonry arch thrusts; and trusses with wood members will generally deform to a greater extent than metal trusses. These

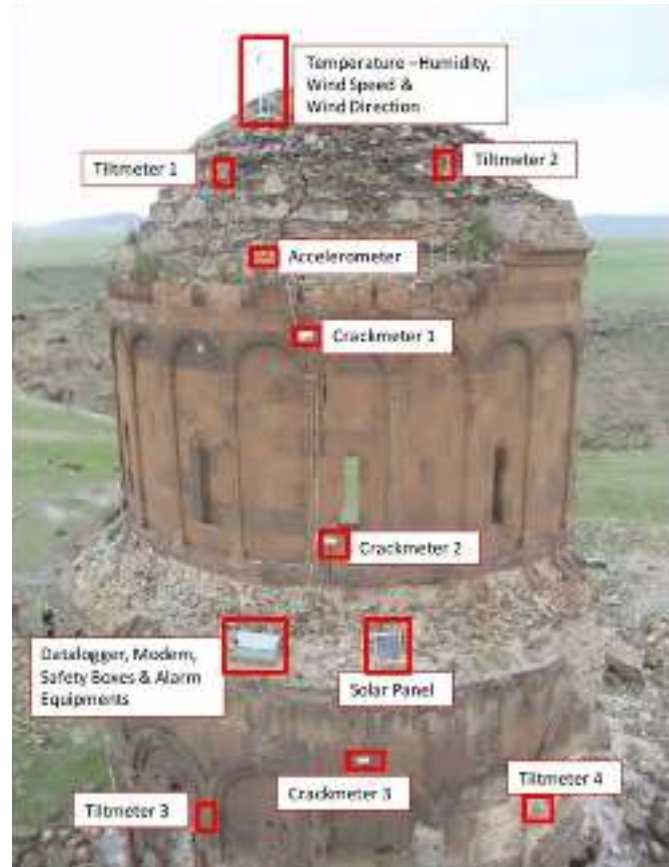


Figure 11. Structural health monitoring of the Church of the Redeemer (1037) at the Ani Archaeological Site in Oçakli, Turkey.

generalizations have been valuable for similar churches throughout the Americas. However, this approach can only provide a probability of the likely response of similar structures and, therefore, is not entirely reliable.

4.9.3. Deductive Approach: Deductive reasoning (top-down logic) is the process of reasoning from one or more general premises to reach a logical conclusion. If all premises are true and the rules of deductive logic are followed, then the conclusion reached is necessarily true.

The approach, which is founded on the scientific method, is the investigative basis for most forensic engineers. A hypothesis is established before or during a rigorous investigation, and the resultant conclusions are derived through deduction. This approach can employ modern methods of laboratory analysis, mathematical modeling, testing of actual materials and components and long-term monitoring. Specific tests can also provide evaluation of safety levels. Examples of such tests include the load testing of elements such as floor assemblies and individual components that are fabricated in the laboratory or the seismic testing of scaled models of buildings on a shake table. However, even with logic that offers a degree of certainty, there are pitfalls — especially with unique older structures. Material characteristics can change with age and use and, thus, be inconsistent within a structure. Masonry decays from exposure; metals fatigue; wood can be attacked by insects and fungi; etc. Inconsistencies between presumed and actual conditions are inevitable due to the simplified representation of the properties of materials and to the imperfect representation of actual structural behavior in mathematical modeling. Scale modeling of actual structures by their dimension and mass, though a precise science, cannot replicate subtleties of that structure. Seismic accelerations and frequencies delivered on a shake table are designed to mimic recorded historic seismic events and deliver only horizontal, not vertical, movements. Therefore, modeling can provide misleading results if applied without understanding its limitations.

4.9.4. Consequently when choosing an approach effective diagnosis is best to adopt two or more of these approaches rather than relying upon a single approach.

4.10. *Safety Evaluation:* Safety plays a key role for the conservation engineer in the diagnosis and determination of optimal therapy strategies. However, the application of modern building codes and standards to heritage buildings is extremely difficult. The application of current safety levels to match those used in the design of new buildings may require excessive, if not impossible, measures that ignore the incalculable robustness of these historic buildings and are also incompatible with preservation goals. As Giorgio Croci, former President of ISCARSAH, once said, each heritage building “should set its own standard rather than be expected to comply with current standards, which have been established for contemporary structures.”

4.11. *Therapy:* There is frequently more than one way to address a structural challenge. Multiple solutions are beneficial, and alternatives should always be considered. A therapy that is sensitive to one structure may be insensitive or even destructive to another. A cost-benefit analysis of alternative interventions should be carried out: costs are measured in the potential loss of fabric due to the invasiveness of the therapy; and benefits measured by the effectiveness of the therapy. By considering numerous feasible alternatives, one can be reasonably certain that strategies that are less intrusive are considered. Some of the ISCARSAH Principles on therapy that are apropos to the discussion are the following:

4.11.1. Therapy should address root causes rather than symptoms.

4.11.2. No actions should be undertaken without demonstrating that they are absolutely necessary.

4.11.3. Any intervention should be in proportion to the safety objectives set, thus keeping intervention to the minimum to guarantee safety and durability with the least harm to historic fabric.

4.11.4. Imperfections and alterations, when they have become part of the history of the structure, should be maintained so far so they do not compromise the safety requirements.

4.11.5. The characteristics of materials used in restoration work and their compatibility with existing materials should be fully established so that undesirable side-effects are avoided.

4.11.6. Where possible, any measures adopted should be “reversible” so that they can be removed and replaced with more suitable measures when new knowledge is acquired. Where they are not completely reversible, interventions should not limit further interventions.

4.12. Once therapy has been enacted and the contract satisfied, it is typical to feel that the work is completed. But conservation is a continuing process that is cyclical rather than linear (see Figure 4). Others have most likely intervened in the past, and their work has an impact on the decisions being made today. It follows that therapy enacted today will have an impact on the decisions of tomorrow. While therapy represents only a brief moment in the history of the building that is more ancient than any human, that therapy may be in-

fluential on the decisions of tomorrow and thus it becomes an awesome responsibility.

4.13. It must be stressed that preventative maintenance is the best therapy. Too often therapy will be performed but then the building will receive a minimum of care until another major work is again required. Lee H. Nelson, an American pioneer in the field of heritage conservation technology, said that “historic buildings would seldom need to be preserved, restored, or rehabilitated [except to update systems] if they were properly maintained.” Deferred maintenance often costs three to five times as much (or more) as preventative maintenance and results in more replacement of materials. Preventive maintenance becomes particularly urgent in seismic areas. During such a disaster, nature has a way of finding the “weakest link” in the form of damage and decay in seconds.

5. Conclusion

The ISCARSAH Principles provide an excellent foundation for engineers to approach heritage conservation and the unique challenge provided by structures utilizing archaic systems. They are in harmony with the building code *ISO 13822:2010: Bases for design of structures - Assessment of existing structures*. The Principles provide a framework to carry out this important work without changing familiar approaches, keeping intervention to the minimum required, retaining the maximum of historic fabrics and providing safety in a way that is in harmony with heritage conservation charters, doctrinal texts and the ISO Building Code on Existing Structures.

Abstract

Historical research, conducted on a building requiring restoration, provides the foundation for analyzing and gaining in-depth knowledge of the building. It allows us to reconstruct its history over the centuries, its modifications and transformations, but also to understand its current pathologies and degradation issues. It is therefore an indispensable tool for competently addressing errors or misunderstandings that could cause irreversible damage to the monument.

Keywords: Historical research, documents, bibliographic sources, archival sources, iconographic sources, literary sources, interpretation

1. Introduction

1.1. What is the purpose of historical research?

Any critically grounded restoration intervention must be preceded by an analytical phase based on thorough historical research, conducted in parallel with other investigations. This concerns both the physical constitution of the building and the factors that have altered its appearance over time. This process is progressive

and requires continuously cross-referencing and critically assessing the information obtained to clarify and reveal the historical process that the building under study has undergone over the centuries.

Historical investigation employs the same tools as scientific research, thus following a rigorous method that starts with the examination and critical analysis of the available historical sources to reconstruct the evolutionary phases, events, and transformations that have occurred over time in the building in question. Such transformations already occur during the conception of a project, including reconsiderations and modifications introduced by the architect or the author during the design stage, as successive refinements leading to the final definition of the work. Modifications also occur – sometimes documented and sometimes not – during the construction phase itself, namely decisions taken on site due to circumstances unforeseen during the design stage.

However, these modifications continue throughout the entire useful life of the building: for functional adaptations, to correct design errors, for improvements, embellishments, to enhance the building's magnificence, or to maintain it in efficient condition. They may also



Ambrogio Brambilla, *Benediction of the Pope in St. Peter's Square* (ca. 1583), before the construction of the new facade, designed by Carlo Maderno at the beginning of the 17th century.



Anonymous, *Procession of Corpus Christi in St. Peter's Square at the time of Innocent X* (1646), with the facade designed by Carlo Maderno and a few years before the construction of Bernini's colonnade began.



Viviano Codazzi, *Exterior of St. Peter's Basilica*, Rome (ca. 1636), with the two bell towers designed by Gian Lorenzo Bernini in the 17th century and demolished a few years later, due to structural problems.



Félix Benoist, *Basilica and St. Peter's Square* (1870), with the colonnade by Gian Lorenzo Bernini.



The Arch of Constantine in Rome, in the mid-eighteenth century, in an engraving by Giuseppe Vasi, before the excavations undertaken in the 19th century to recover the original levels.

involve transformations that alter the building through incongruous and incompatible uses, erase its original meaning, or adapt it, due to a lack of resources, to different versions from its original state of splendor.

Historical research also guides in identifying the architects, engineers, and their collaborators, as well as the clients and promoters of a work, in addition to the contractors and companies involved in the execution of its parts: structures, systems, details, and finishes. Moreover, it allows discovering changes in mentality across different historical periods, together with the intentions and aspirations of those who promoted the work – whether aimed at self-representation, acts of munificence, or simply at addressing functional purposes and needs for services.

It is also aimed at clarifying construction techniques and systems, as well as the materials employed in building construction: ingenious and original solutions,



Constantine's Triumphal Arch in Rome, in an oil painting by the German painter Wilhelm Oswald Gustav Achenbach (1886).

in some cases now obsolete, which illustrate the stages in the evolution of human ingenuity.

This includes how different materials were used, both in their natural state and as products resulting from manufacturing processes that endowed them with properties and qualities suitable to ensure durability over time.

By progressively expanding the sphere of knowledge, research may extend to the study of the places where such materials were extracted or obtained: mines of sand or aggregate, quarries of marble, granite, limestone, sandstone, or more precious materials such as alabaster and onyx.

It also involves understanding production methods, such as those related to lime – used in mortars, plasters, and finishes – and, consequently, the kilns in which limestone was processed and fired to produce quicklime, as well as the pits used for slaking over



The arts and crafts of the Mexican natives, on a page from book 10 of the *Florentine Codex*, compiled by Franciscan friar Bernardino de Sahagún in the mid-16th century.

extended periods to improve its performance and characteristics.

The same applies to clay, widely employed in the production of bricks, terracotta elements, and roof tiles, as well as in the manufacture of glazed ceramics, extensively used in Middle Eastern cultures.

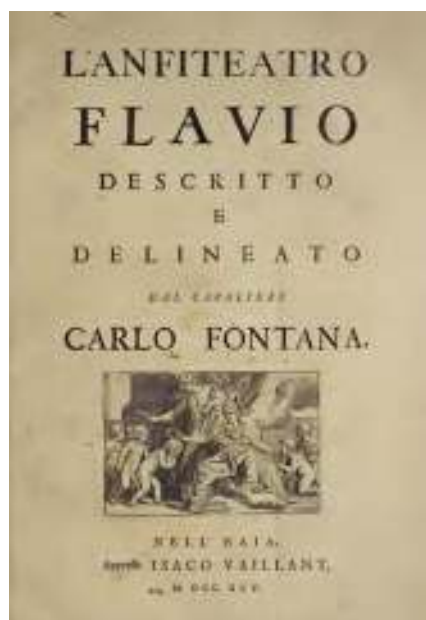


Advertisements and catalogues of new materials produced in the first half of the twentieth century in Italy.

A different discourse has to be done for materials patented during the twentieth century: in the race to outperform competitors, these were widely promoted both in specialized technical and industrial journals and in interior design magazines. As expressions of the taste and fashion of their time, these materials were rapidly adopted but soon fell out of production, as they were replaced by other products offering superior performance.

For such materials, brochures and advertising inserts, together with contemporary journals, provide abundant practical information, allowing their characteristics to be understood in detail, since most of them are no longer available on the market today.

Historical research focused on the building also requires the study of craftsmen's work – almost



Frontispiece of ancient books on the buildings of Rome. The first, by Carlo Fontana, on the *Flavian Amphitheater*, published in The Hague in 1725; the second, by Giovanni Marangoni, also on the *Flavian Amphitheater* (Colosseum), published in Rome in 1746; the third, on the buildings of modern Rome by Jean Barbault, published in Rome in 1763.

disappeared from the construction site – as they carried out specialised activities, both in terms of technical complexity and refinement of details and finishes. The preservation of this intangible heritage – also referred to as know-how or crafting expertise – is indeed a fundamental factor for competent intervention on historic buildings. Consequently, any description related to this craftsmanship in publications or documents represents an asset to preserve, as important as the monuments to protect.

Why gather all this information? First, to identify and properly assess the building's value, uniqueness, and the originality of its design and technical solutions. Second, to inform intervention strategies: to avoid harming the historic fabric through insufficient understanding of its specific characteristics, and to develop

conservation and restoration measures that are materially compatible and conceptually coherent with the historic building. Ancient measurement systems – Roman palms, the Florentine *braccio*, the *Vara castellana*, among many others – offer valuable clues for understanding how buildings were dimensioned and how their proportional relationships were conceived. Measured today in the decimal metric system – or in English units – those dimensions can appear irregular or arbitrary; yet it is clear that they once corresponded to coherent traditional practices of measurement and surveying.

In some cases, moreover, they embodied proportional relationships derived from mathematical, geometrical, or astronomical studies, such as the golden ratio, already known in the time of Pythagoras and Euclid.



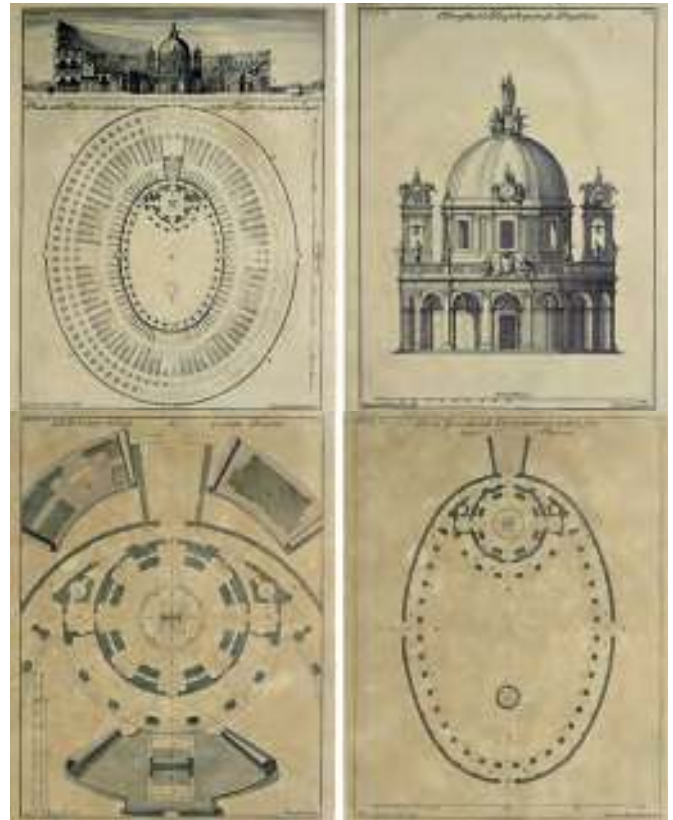
One of the oldest representations of the Colosseum in Rome, on a sestertius minted between 80 and 81 AD, at the time of its inauguration.



Detail of the map of Rome by Giovan Battista Falda, from 1676, with the Colosseum at the centre.



The Colosseum in Rome in a drawing from the Codex Escurialensis (ca. 1480).



Carlo Fontana, project for a church dedicated to the Holy Christian Martyrs, to be built in the arena of the Colosseum (late 17th century).



Carlo Lucangeli, Wooden model of the Colosseum (1790 and 1812).



Illustrated postcard with a photograph of the Colosseum, 1928.



Luigi Maria Valadier, survey of the hypogea of the Colosseum, commissioned by Emperor Napoleon Bonaparte in the early 19th century.



Illustrated monthly supplement of the periodical "Il Secolo", with a page dedicated to the Colosseum, June 25, 1887.



Luigi Gallandt, The Colosseum in a micromosaic (mid-19th century), conserved in the Golestān Palace in Tehran.



Map of the Tiber River between Rome and its mouth in the Tyrrhenian Sea. The survey, conducted by Bolognese engineers Gambarini and Chiesa in 1744, shows the river's course and the depth of its bed.

2. Where to start?

What, then, are the principal sources on which historical research is grounded, and where should one begin the orderly and systematic path we have set ourselves, if we are to understand an architectural artefact in all its features and specificities?



Gregorian cadastre (first half of the 19th century), the Trastevere district of Rome. Existing buildings are in red. Courtyards, gardens, and undeveloped spaces are in green.

2.1. Bibliographical research

It is useful to start with a bibliographical survey. Even when it appears that a building has never been studied, that assumption often proves mistaken. In every period, there have been individuals interested in the past, intent on retracing the stages of the historical



Rome Master Plan, approved by the City Council in 1882. In red and pink, all the new neighborhoods that will arise when Rome becomes the capital of United Italy, for the construction of residential and office buildings.

development of places and their buildings. Their studies represent the foundation for the knowledge base that has gradually been transmitted from generation to generation, leading to broader accounts with wider temporal scope and geographical range. Those scholars relied on local sources, including docu-

ments that may have disappeared – destroyed by catastrophic events such as fires, floods, or earthquakes – or intentionally removed. In such cases, we are only left with what they recorded in writing. Their texts, however, also help us identify the archival collections that are most worth exploring.

2.2. Archival research

Archival research may be hard to approach without some form of guidance – often provided precisely by those who have ventured into those uncertain and frequently tortuous paths before us: these routes may sometimes lead nowhere, but, at their best, prove thrilling because they bring us directly to the information we seek.

One point, however, must always be kept firmly in mind: every piece of news, every datum, every item of information derived from archival or bibliographical documentation must be subjected to critical analysis. It must be compared with other documents and other sources; it must be verified – both for authenticity and for reliability – and, above all, it must be tested together with the building itself, to check the correspondence between what texts convey and the preserved building fabric.

2.3. Iconographic research

Alongside archival and bibliographical research, iconographic investigation likewise takes a crucial role. Representations of the building across different historical periods provide unequivocal evidence of transformations or, conversely, of the continuity in its condition: alterations, additions, partial demolitions. These include cartography and maps, drawings, paintings, sketches, photographs, surveys and engravings. They also reveal the role the building has played within its setting over the centuries: whether it was valued or neglected, and whether it formed part of the social, economic, religious, or political life of the city.

A careful examination of iconographic sources can disclose details that escape direct observation: features

that no longer exist, changes for which no written record survives. For this reason, they are among the most powerful tools for understanding the architectural object in its historical evolution. As the saying goes, «an image is worth a thousand words»: even a newspaper clipping or a faded tourist postcard may help solve uncertainties concerning a date or an otherwise undocumented transformation.

In this group, we can also include scale models of buildings, built in the past with the aim of making the client understand the characteristics of the building, but also to encourage him to carry out the work.

2.4. Economic and construction documents

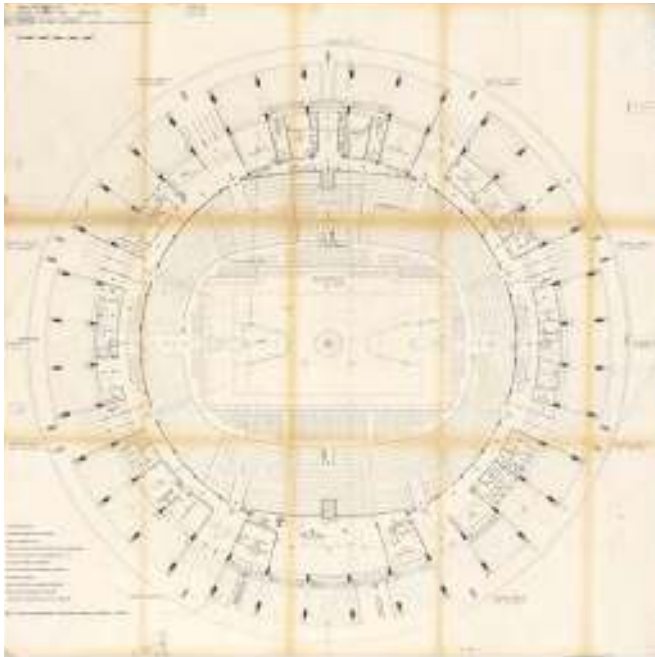
Less appealing – sometimes even tedious, yet no less essential – are economic and construction documents: contracts, specifications, bills of quantities, measurements, and cost estimates.

Their importance lies in the fact that they describe, often with the greatest degree of accuracy and reliability, the works actually carried out, interventions that can frequently be verified through a careful inspection of the building fabric.

2.5. Literary descriptions

Equally valuable, from a different perspective, are the accounts and descriptions left by explorers and travelers in their journals and diaries.

These sources record aspects of everyday life, customs, and practices associated with the building, revealing how the monument was experienced at specific moments in time, as well as the impressions and perceptions that convey its deeper significance for the society of each period.



Plan of the Palazzetto dello Sport in Rome, by Pier Luigi Nervi and Annibale Vitellozzi (1956-57)

2.6. Historical research is like a police investigation

While many buildings are characterized by an abundance of historical documentation accumulated over the centuries, others – less known or located in remote and rarely visited areas – do not experience the same fortune.

In such cases, historical investigation takes on the character of a detective inquiry, with the «body of evidence» being the architectural artefact itself. The building must tell its own story: the struggles it has endured, the wounds it bears, the events it has witnessed.

Its material substance is the sole source available for



Photograph of the construction site of the Palazzetto dello Sport, by Pier Luigi Nervi and Annibale Vitellozzi (1956-57), built for the 1960 Olympics.

understanding and reconstructing its history.

For this reason, as already noted, a thorough knowledge of construction techniques, materials, and their modes of use and deterioration is essential.

Only through comparison with other contemporary buildings – whose histories are better documented – hypotheses can be made regarding dating, the processes that have affected conservation or decay, and the broader circumstances that shaped the building.

As with every product of humankind, such analysis ultimately reveals ingenuity and intelligence, the capacity to solve concrete problems, and the solutions that made those buildings unique and irreplaceable – these are the qualities that now lead us to recognize them as cultural heritage.

Documentation Methods, Geometric Studies, Mapping the Damages and Simple Non-Intrusive Diagnostic Techniques

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Abstract

Two main nowadays documentation methods – laser scanning and photogrammetry – are presented. Advantages, drawbacks and accuracy of both methods are discussed on the basis of measurements performed by the author on objects of various type and scale, beginning from an ornamented leather cover of a Turkish book to masonry dome. Two laser scanning technologies are discussed. One is a widely known LiDAR scanning used to document the geometry and actual condition of large objects in-situ. The second one is a laser triangulation method useful for laboratory measurement of the shape and surface details of small heritage objects and paintings. In the case of several objects photogrammetry documentation campaign was performed at the same time as laser scanning. This allowed to present a comparison of the results obtained with the laser scanning and the photogrammetry. Scans of the larger objects, also for the purpose of deformation monitoring are presented on a literature basis. A possibility of using photogrammetry to monitor masonry degradation processes is also discussed.

Among simple non-intrusive diagnostic techniques application of surface waves to testing stone, brick and masonry walls is presented. Also a possibility of using Smartphones to

diagnostic and monitoring is shortly discussed. Measurement of masonry material strength properties using Compact Diagnostic Test (CoDiT) procedure is shown as an example of a minor-destructive testing.

Keywords: Laser scanning, Photogrammetry, Point Cloud, Documentation, Masonry, NDT & MDT testing, Compact Diagnostic Test CoDiT, Rayleigh wave, smartphone

1. Documentation methods

Methods which have been in use since centuries like tape measurement or methods which have tens or more years of application like photography, geodetic measurements using total station equipment and newer methods like hand operated ultrasonic and laser distance meters are still in a wide use. Especially high resolution (HiRes) digital photography, laser and tape distance measurements are commonly used for quick evaluation of observed object condition and recording its state. Probably this will not be changed in the future thanks to the portability and low cost of equipment and its ease of implementation on site.

These three methods give only 1D or 2D documentation



Figure 1. Coded and non-coded targets used in photogrammetry and laser scanning.

data which is not enough for proper maintenance, management, conservation planning and engineering restoration works. This is the reason why full-field 3D documentation methods have been developed.

For a purpose of heritage objects 3D documentation terrestrial lasers scanning and photogrammetry are the most promising methods. The last one has its variations like terrestrial with free hand and tripod camera holding [1] and a car equipped with a high mast [2], or aerial photogrammetry with kite, balloon [3, 4], drone or plane [5] used as a camera carrier;

Here one thing should be pointed out. The 3D documentation methods need to be often supported and enhanced with 1D distance measurements, sometimes in several directions, for calibration, verification and accuracy evaluation purposes. These are either direct pointing of the points distance in photogrammetry or using coded targets with known dimensions for automatic calibration and coded and non-coded targets (Figure 1), e.g. reference spheres in photogrammetry [6] and laser scanning [7] in a case of large and complicated objects where e.g. combination of room models and 3D shapes of building facades are together forming



a larger model. In such a case precisely measured targets positions and distances are also necessary to “glue” individual 3D subspaces into a one model space.

2. Accuracy considerations

2.1. Time of Flight Laser Scanners - TOF (LiDAR)

Distance from scanner to an object is calculated on a basis of measurement of the time the laser light passes to the object and reflected from its surface returns back to the scanner detector. Time measurement accuracy used in laser scanners is $10^{(-11)}$ part of a second which gives a distance measurement resolution of approx. 3 mm.

If additionally phase detection is used then the time reading resolution is 10 times better ($\text{time_res} = 10^{(-12)}$ part of a second) and the distance measurement resolution is 0.3 mm. This is the highest possible theoretical accuracy of available TOF scanners. In practice the accuracy is lower than the theoretical resolution and depends on the distance to a scanned object. For short distances of several meters the achievable real accuracy

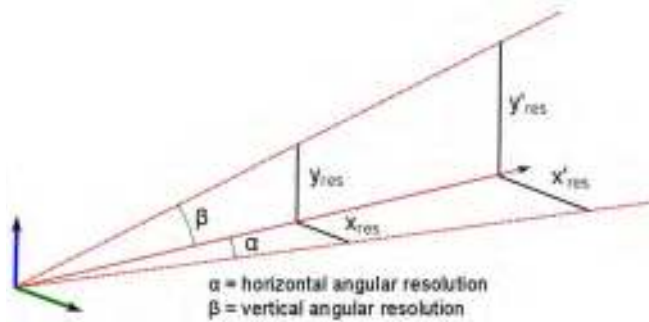


Figure 2. Angular and spatial resolution.

of distance measurement is usually about 0.8 mm. Another resolution of LiDAR is a spatial resolution which is linearly dependent on the distance from the object to the scanner. This is a consequence of scanning with a constant angular resolution which is illustrated in Figure 2.

Scanners Surphaser 25HSX ER-XS and 100HSXER have angular resolutions $\alpha = \beta = 40$ arcsecond. Spatial resolution of these scanners in x and y directions are given in Table 1 for three different distances between an object and a scanner.

Laser-object distance	x and y resolution	Number of points per cm ²
10 m	> 1.95 mm	26
15 m	> 2.9 mm	12
40 m	> 7.8 mm	1.6

Table 1. Example of scanning spatial resolution for angular resolution of 40 arcsecond.

Scanner FARO Focus 150 has angular resolutions $\alpha = \beta = 19$ arcsecond. Even with such a two times higher angular resolution small or narrow details (cracks) often cannot be registered.

Accuracy of the measurements depends also on light propagation conditions. Any air turbulence due to

local temperature differences on the light travel path causes refraction of the laser beam and its path becomes not a straight line. The light reflects from the object surface point which has different coordinates than that recorded on the basis of the angular position of the laser scanning head.

Manufacturer *declared accuracy* is based on 1 sigma confidence bound which means that 68.2% of points is registered with an error smaller than the statistical deviation.

To make sure about the *real accuracy* one needs to calculate coordinate differences for several scans of the same object made from various scanning positions. Illustrations of the problem and correct procedure are given in Examples 2 and 4.

2.2. Photogrammetry

Theoretical accuracy of photogrammetric measurements based on number of pictures taken with one camera is equal to [8]:

$$\sigma_c = \frac{m_f \sigma_i q}{\sqrt{k}}$$

where:

σ_c – model accuracy [mm];

m_f – photogrammetric scale (object distance/real focal length, = image demagnification);

σ_i – accuracy of measure in the image [mm];

q – coefficient depending on configuration (0.4 for highly convergent photos, up to 0.7);

k – number of pictures used to build 3D projection of the registered point.

Another useful estimation is given in an information

Camera	Pixel Count	Precision	50' Object	15m Object
Photo CD/ 35mm Film	1536 X 1024	1:1100	0.5"	15mm
High Quality Digital	4500 X 3000	1:3100	0.2"	5mm
High Quality Digital w/ Targets	4500 X 3000	1:30000	0.02"	0.5mm
Mid. Quality Digital	2048 X 1536	1:1400	0.4"	10mm
Low-End Digital	640 X 480	1:450	1.33"	33mm
Video Camera	500 X 375	1:350	1.7"	43mm

Figure 3. Expected precision of photogrammetric measurements elaborated with PhotoModeler Pro software [9].

available for PhotoModeler software cited in Figure 3 [9]. The estimation shows that using targets placed at the photographed scene increases model accuracy about 10 times. Similarly the better the camera resolution the higher the accuracy. This is related to " σ_i – accuracy of measure in the image" in the above theoretical estimation.

2.3. Triangulation laser scanner EPPM-I

The triangulation scanner EPPM-I is dedicated to small objects scanning [10]. It has a working area of approx. $300 \times 400 \text{ mm}^2$. Its measurement resolutions are: in X,Y direction (scanning points distance) $> 0.05 \text{ mm}$; in Z direction (perpendicular to the scanner surface) $> 0.0005 \text{ mm}$.

Accuracy of Z distance measurement is lower due to micro-vibrations (of both painting surface and traveling laser head) and non-ideal flatness of laser head guiding system.

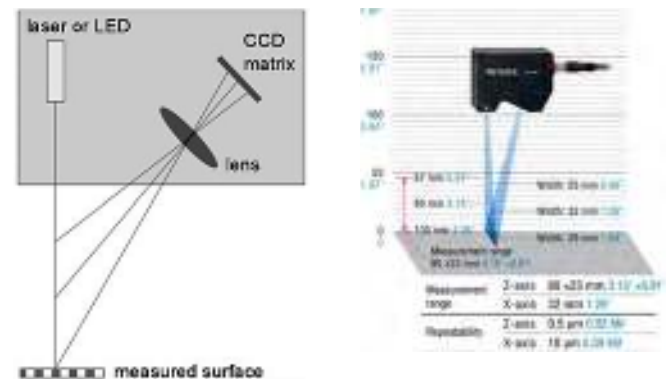


Figure 4. Triangulation working principle and range parameters of the used laser head.

Tested accuracy of Z distance measurement is 0.08 mm. Measurement principle is shown in Figure 4. A "sheet of light" is projected onto the object surface and bright points positions are recorded with a 2D sensing element built into the laser head. Position of the bright points image on the sensing element depends on the distance of the head to the points on the scanned surface. The method is sensitive to abrupt changes of surface normal directions which generate additional source of errors.

3. Documenting examples

Cultural Heritage objects vary largely in size from several square millimetres (pieces of archaeological objects, small coins) to square kilometres in case of large sites and historic urban areas.

Even if we leave apart documentation of very small objects in our discussion we are still facing the situation when a documentation method must be individually chosen for the object or a class of objects.

Examples presented here are selected to cover objects size from 0.01 m^2 to 1 km^2 (Figure 5) that is



Figure 5. Documentation method depends on the *scale* and the *purpose*.

they vary in a scale as $1:10^8$.

Documentation of heritage objects is made for various purposes like e.g. management, restoration, mapping of material composition, basis for FEM computational model etc. The documentation purpose also influences, mainly through required accuracy, the selection of the documentation method.

The presented examples of 3D documentation using LiDAR, photogrammetry and laser triangulation scanner cover various purposes of documentation elaboration.

3.1. Example 1: Schönbrunn Palace and Park in Vienna, Austria

Object size = approx. 1 km^2

Documentation purpose => Maintenance and Management

Place: in-situ

Method: TOF laser scanning

Schönbrunn Palace (Figure 6) was built in 18th century

and its large park being a former hunting area was decorated with statues, sculptures, walks and avenues, fountains, orangery, Roman Ruins, Gloriette (Figure 7) and other objects. There are also several other historical buildings used previously for various purposes.

Management and assuring a proper maintenance of such a large, historically rich and diversified heritage site requires the documentation of all the objects including their details with at least 10 mm accuracy. As a documentation method laser scanning was chosen.

The task was accomplished by Centre for Digital Documentation and Visualisation CDDV, Historic Scotland, Glasgow School of Art. Every piece of objects in the Palace, Gloriette and other buildings interiors was scanned as well as the buildings architecture and Park avenues, statues, fountains and the others [11].

A very interesting and instructive concept of managing and using the documentation data was deployed. The 3D models were not immediately built. The original data are stored by the scanning company and are used



Figure 6. Schönbrunn Palace Building.



Figure 7. Gloriette at Schönbrunn park.

to build models e.g. of the main staircase when engineering works were planned and conducted. Then after the work was completed the new scan of the staircase was done to assess and evaluate the differences between original and after-works geometry. The same idea, of local modelling when necessary, is applied in other cases like for analysis of possible existence of open channels hidden inside walls which may

increase a fire risk for the heritage structure.

This dramatically lowers the maintenance cost as an elaboration of the scanned point cloud data and building 3D models is the highest cost of the documentation. And this cost is avoided until a model of some part becomes necessary.

At the same time full and very precise documentation data is recorded at reasonable cost to be used when needed.

3.2. Example 2: Gothic church in Ząbkowice Śląskie, Poland

Object size = approx. 350 m²

Documentation purpose => Finite Element Modelling, structural strengthening and post-intervention monitoring

Place: in-situ

Method: TOF laser scanning

The church was built in the second half of the 14th century. The presbytery vault was completed in 1566 after



Figure 8. Ząbkowice Śl (courtesy of Łukasz Bednarz, ISCARSAH, Poland).

burning down of its previous timber roof during Hussite's war in 1428. The vault was restored in the neo-Gothic style in the years 1893–95 using machine-made bricks, smaller than the original ones.

Within time brick vaults and walls cracked and repair works became necessary. Prior to the works Finite Element Model analysis was performed on a basis of geometrical models (Figure 9) developed using 3D high resolution laser scan data (in 2009).

Steel rods, Carbon Fibre Reinforced Polymer and Carbon Fibre Reinforced Cementitious Matrix technologies were used to consolidate and strengthen the walls and vaults of the presbytery.

Then the same scanning and 3D modelling procedures were used after the repair works (2011) and the next one after two years (2013) from the previous scan.

This was done for a purpose of deformation monitoring and evaluation of repair works effectiveness [7].



Figure 9. Ząbkowice Śl. Church vault model based on a laser scan (courtesy of Łukasz Bednarz).

Year of scan	Laser scanner	No. of scans	Resolution	Statistical errors [mm]				Spacing of points [mm]
				Average	Deviation	Min	Max	
2009	FARO Photon 120	3	6.283 mm/10 m	2.2	1.7	0	5.4	10
2011	FARO Focus 3D 120	3	6.136 mm/10 m	2.5	1.5	0	5.3	10
2013	FARO Focus 3D 120	3	6.136 mm/10 m	2.9	1.2	0	5.5	10

Table 2. Resolution and accuracy of scanned points of church vault.

Parameters and the accuracy of scans of the presbytery in the years 2009, 2011 and 2013 are presented in Table 2 based on [7].

In each scanning campaign there were taken three scans of the same vault from different scanner positions and statistical analysis was performed to assess the accuracy of the laser measurements.

3.3. Example 3: Blessed Ladislaus Chapel Dome of St. Anne's church in Warszawa, Poland

Object size = approx. 50 m²

Documentation purpose => repair, restoration and conservation

Place: in-situ

Method: TOF laser scanning, photogrammetry

The previously gothic church subjected to several destructive fires was rebuilt at the beginning of XVII century



Figure 10. St. Anna's church, scanning of the chapel dome, (left photo courtesy of Katarzyna Górecka, Academy of Fine Arts in Warsaw, Poland).

and decorated in XVIII century. A documentation of the dome of a side chapel (Figure 10) was prepared using laser scanning and two photogrammetric models based on ambient light and artificial light photographs. The 3D model from the laser scan is used for drawing plans and cross-sections and as a reference for photogrammetric models to evaluate their accuracy. The first photogrammetric model based on ambient light and images having 12.8 megapixels has a mean error of 0.036 m when compared to the laser reference scan. The second model based on artificial light and images having 50 megapixels made after the dome conservation has a mean error of 0,0046 m. This is a very good agreement with the reference laser scan model. It shows that photogrammetry of well texturized objects can be used for 3D modelling as a first choice method as it offers high accuracy plus better texture and damage mapping than do laser scans [12].

3.4. Example 4: Large Curvilinear Canvas Painting in Namur Cathedral, Belgium

Object size = approx. 17 m²

Documentation purpose => strengthening, conservation and post-intervention monitoring

Place: in-situ

Method: TOF laser scanning, photogrammetry

Hans Mauritius Loder painting "Adoration of the Magi" is technically interesting because it is a large concave canvas painting with a shape roughly adjusted to a rounded apse wall of the church (Figure 11). Canvas sagitta is more than 34 cm and the canvas heights is 370 cm with the length of 450 cm.

Improper stretching of the canvas and its creep resulted in several creases and a large bulge of 18 cm in a lower part of the canvas [13].

At first three scans were taken in Sept. 2012 to assess modelling accuracy, to document painting before conservation works and to build models of two tables (concave and convex) planned to be used during conservation of painted and reverse sides of the canvas. Histogram presented in left part of Figure 12 shows that points within about 2 Sigma (95,5%) confidence bound have their distance to the reference point-cloud



Figure 11. St. Aubain Cathedral in Namur, Belgium. Left, scanning of the curvilinear painting “Adoration of the Magi”. Right, scan preview with visible two holes in the upper part of the painting.



81 mm. But knowledge about the situation in-situ allows to give the correct explanation. The reason is that there exist two holes in the canvas painting and some scanned points belong to the wall behind the canvas. These two holes when scanned from three different angles (in reference scan from a scaffolding at the canvas level and in the other two from two different

lower than 2 mm. More than 1 500 000 points (60%) have position error lower than 0.65 mm, but more than 80 000 points (3%) have position error greater than 2 mm.

Analysis of the rather low accuracy results supported with statistics of distance computation for two scans compared with the reference one given in right part of the Figure 12 does not explain why maximum differences in measured distance are as large as 34 or even



Figure 12. St. Aubain Cathedral in Namur, Belgium. Left, histogram of the point distance differences between two scans. Right, statistics of distance computation for two scans compared with the reference one.



Figure 13. Ornamented leather cover of the Turkish book. Figure 14. Two pages of the Turkish book.

scanner placements on the floor) transmit the light to different wall points. Hence distances are measured to various points and from various angles. This part of information is false and irrelevant to the information about painting shape. It is important to stress that automatic (statistically based) removing of this false information removes also some important information about the canvas bulges and folding. So a conservator/restorer should always be suspicious about nicely cleaned 3D models and justify the model correctness using the whole available knowledge. The fourth scan was done after repositioning of the conserved and strengthened canvas on the wall in November 2014. The fifth scan was done in Aug. 2015 for post intervention monitoring purpose to check whether a global shape changes appear or not [14, 15]. The local deformation process was monitored with SASMM (described in “Simple Non-intrusive Diagnostic Techniques” section) using image correlation method developed in 2006 for deformation measurement of historical constructions [16].

3.5. Example 5: Leather cover of old Turkish book

Object size = approx. 0.015 m²

Documentation purpose => conservation, restoration

Place: laboratory

Method: triangulation laser scanning, photogrammetry

This old Turkish book is owned by Library of Warsaw University.

Conservation project undertaken at Faculty of Conservation and Restoration of Works of Art at Academy of Fine Arts in Warsaw included restoration of a missing leather cover of the book. The existing cover had a pressed-in ornament shown in Figure 13. The idea was to copy the ornament on a newly made missing cover replica. For this purpose manufacturing of a stamp being a reverse of the shape of existing ornament was assumed.

To facilitate the stamp manufacturing an attempt of producing a geometrical model of the ornament was undertaken using triangulation laser scanner and super close range photogrammetry.



Figure 15. Photogrammetric model of the cover ornament.



Figure 16. Model of the cover ornament based on the laser triangulation scanning.

Photogrammetry model was built on a basis of free hand images taken ad hoc with smartphone Samsung S-4 camera. It allowed to obtain a point cloud of 34 718 points and a mesh with 68 725 faces. The result is illustrated in Figure 15. Using triangulation scanner EPPM-1 a point cloud of 479 969 points was recorded and a mesh with 959 060 faces generated which gives aver-

age of 120 faces per 1 mm². This demonstrates the technical feasibility of generating highly detailed models of small objects of art. Fitting a normal distribution to point height differences between the scan model and photogrammetry model of the cover ornament gave a remarkably good mean value 0.006 mm but with much higher standard deviation equal to 0.464 mm.

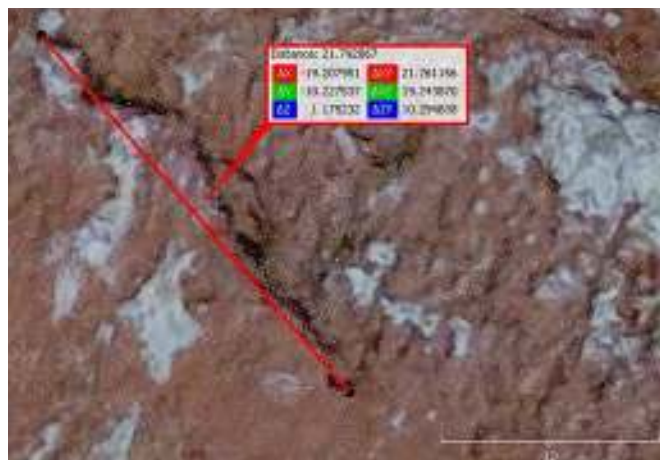


Figure 17. Hand-held camera photogrammetric model of a brick surface (left). Crack points coordinates shown in Cloud Compare (right).

It is known (section “Accuracy considerations”) that triangulation scanner offers greater accuracy of distance measurement than photogrammetry but on the other hand is susceptible to errors related to abrupt changes of normal to the scanned surface. Manual cleaning of laser point cloud would eliminate spurious peaks visible in Figure 16 and improve the agreement between both models. The other improvement would be application of a high quality DSLR camera instead of a smartphone one. However probably the both obtained models could be accepted for the leather cover restoration purpose as the final ornament shape would depend also on the leather elasticity, stamping technology applied and stamp manufacturing quality which all introduce their specific inaccuracies influencing the final ornament restoration result.

3.6. Example 6: Possibility of deterioration monitoring

Object size = approx. 0.01 m²

Documentation purpose => deterioration monitoring

Place: in-situ

Method: super close range photogrammetry

A mural painting inside the Etruscan *Tomb of the Monkey* (*Tomba della Scimmia*, 480-470 B.C.), located at Poggio Renzo Necropolis was photographed with Canon EOS 400D (10 megapixel) + EFS 60 mm macro lens using motorized stand for lateral translation of the camera. Digital model of each tested fragment of the wall surface was developed based on three images taken in parallel configuration of the camera. The same wall fragments were documented photogrammetrically after 8 months. No meaningful changes of the surface shape were detected [17].

The authors used a dedicated software so the camera configuration could not be changed to a skew one. Nowadays there exist software packages (free and commercial) which allow to produce photogrammetric models from free hand taken images or even a collection of movie frames. The software can successfully be used for generating 3D models for super close distances i.e. several centimetres. As it was discussed in



Figure 18. St. Anna's church, mapped real cracks and a crack imitation by a wire, (courtesy of Katarzyna Górecka, Academy of Fine Arts in Warsaw, Poland).

"Accuracy Considerations" section of the paper an expected accuracy can be within an order of 0.01 mm for such a close distance. It is worth of testing whether such free hand images and generated models can be used for surface deterioration studies.

A simple ad hoc example of XIX century brick modelling using super close photogrammetry is given below.

3.7. Example 7: Ad hoc photogrammetric example

Object size = approx. 0.01 m²

Documentation purpose => deterioration monitoring



Figure 19. Salt efflorescence recorded by laser scanning in orthodox church in Chevetogne, Belgium (scan author).

Place: laboratory, possibly in-situ

Method: free hand super close range photogrammetry

This example (Figure 17) illustrates a feasibility of using the super-close range photogrammetry to surface shape studies but also that a texture applied to a model is extended to the whole model surface, while real 3D information exists in model vertices only. The 3D data apparent continuity is usually obtained through triangulation procedure based on the registered points data. In surface deterioration studies approximation of the surface needs to be carefully analysed as it introduces model smoothing which may screen subtle degradation changes. The same regards canvas painting and tapestry surface documentation.

4. Mapping of damages

4.1. Mapping of 1D damages

Mapping of 1D damages (cracks) must usually be done manually on HiRes photos or in CAD drawings (Figure 18). A fragment of 3D photogrammetric model of the

surface of the dome from Example 3 shows manually mapped cracks and two lines which are not cracks at all. One of them is a steel wire hanging from the dome lantern, used for window opening manipulation and the second line is a shadow of this wire. This illustrates that there exist a limited possibility of automatic mapping of damages in photogrammetry modelling.

4.2. Mapping of 2D damages

Mapping of 2D damages (salt efflorescence, fungi, damp) can be documented automatically by laser scanning and photogrammetry thanks to a change in colour or absorption/reflection properties of a surface (Figure 19).

Another example showing that mapping of damages with laser scanning cannot be in some cases correctly done is related to post-intervention monitoring of the curvilinear painting described in Example 4.

In Figure 20 there are drawn three horizontal cross-sections of the painting. Each of them shows the local shape of the painting recorded with scans of the original state before conservation in year 2012 (blue colour), the shape just after conservation and repositioning of the painting on the wall in year 2014 (red colour) and the shape after next eight months in year 2015 (green colour) [14].

The obvious result is that no deformation appeared during these eight months but the scans show another interesting situation. All three scans show large discontinuities of the recorded shapes. In the first scan this is a large hole in the painting and in the scans taken after conservation this hole is repaired. There are however two other “hole-like” looking parts in the repaired painting in totally new locations. This misleading observation comes from

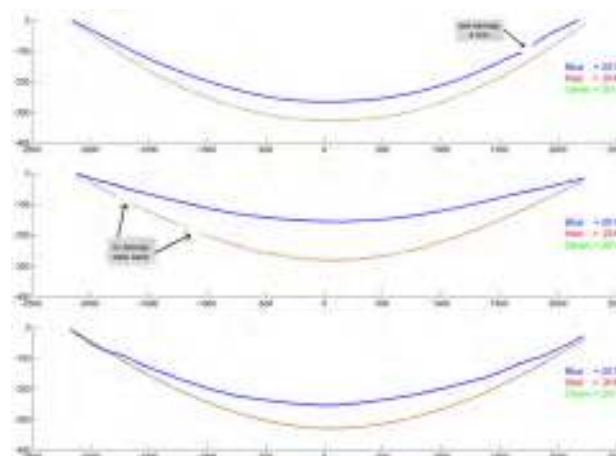


Figure 20. Three horizontal cross-sections from LiDAR scans of the painting in St. Aubain Cathedral in Namur: before restoration (2012), after restoration (2014) and deformation monitoring result (2015) (upper). Crack points coordinates shown in Cloud Compare (lower).

the fact that no laser light reflected from the painting was recorded by the scanner for these locations because the painting surface was absorbing the light due to its very dark composition there.

The conclusion is that automatic discrimination between a hole and a highly absorbing surface was not possible even in a single object.

4.3. Mapping of interventions

Mapping of interventions must usually be done manually on HiRes photos or in CAD drawings as it was presented in [12] in the case of previous interventions documentation made for conservation works on the chapel dome in St. Anna's church in Warsaw. In some cases photos and photogrammetry can map the intervention areas if there is distinguishable difference in texture which was presented in the same [12] paper in a case of documenting new interventions prior to repainting the reconstructed layer.

5. General remarks regarding documenting methods

Documentation method depends on the *scale* and the *purpose*.

5.1. Photogrammetry

- Photogrammetry is more universal – can be used at super-close range (even in microscope) and for a very large town and site areas. The best equipment are full-frame mirror-less cameras.
- Registered models are already texturized to the best possible accuracy.
- Photogrammetry is usually less efficient while precisely documenting inner and outdoor space for integration into a single large model as calculation of the data needs higher computational cost than in the case of comparable laser scans. It is efficient for large models when only coarse geometry (i.e. low resolution) documentation is produced (e.g. producing CAD drawings without artefacts details).
- Using a camera on a high mast is probably the first choice from operational point of view in documenting high objects like mosques and minarets in towns where you cannot fly drones or balloons and if top part of the structure is invisible for scanners.
- The same “camera on mast” seems to be the best for quick documentation campaigns in countryside and off-road areas. It can be used in weather conditions when drones cannot fly and it uses HiRes large frame cameras offering better accuracy than drone cameras.
- In the case of archeologic sites sometimes it is necessary to avoid placing the scanner inside the area. In such a case using drones and photogrammetry is a better choice. The same is e.g. at a cost-line areas when

documenting a rock slope together with heritage object is necessary.

- Developing of an efficient method of free-hand photogrammetry to monitor surface deterioration is worth to be discussed.

5.2. Terrestrial Laser Scanning

- Laser scanning is faster and much less laborious. The best are scanners integrated with HiRes camera which give the texturized models.
- Terrestrial scanners can be used for objects of many square kilometre area down to about 1 m². At large size range they give greater accuracy than photogrammetry. For smaller objects a change of the scanner type (triangulation, structured light, handy scanner) or using photogrammetry is better.
- Large and complicated objects require a lot of scanner positions and registration of many Scan Worlds (interiors, outside parts of a monument, artefacts scanned with handy scanner). These Scan Worlds can be more precisely merged when special fixed targets are additionally used.
- Whenever possible make full point-cloud documentation in large projects but build partial models locally at the time when you need it.
- It does not make sense to document for conservation purpose e.g. Persepolis in the one mesh model. But it is reasonable to have its point-cloud representation down to the required detail to be able to generate any local mesh whenever it will be necessary.

5.3. Both scanning and photogrammetry

Both scanning and photogrammetry are susceptible to surface condition:

- scanners may not work well on dark, wet or light absorbing surfaces (e.g. Neolithic rock painting in a cave);
- photogrammetry does not work on flat surfaces without texture and may not be useful for highly reflective surfaces (e.g. mosaic tiles).

5.4. Combination of scanning and photogrammetry

Combination of scanning and photogrammetry offers the highest geometrical and contextual (materials, textures, damages, repairs etc.) accuracy and quality.

6. Simple non-intrusive diagnostic techniques

6.1. Surface Rayleigh wave (V_R) measurement

Surface Rayleigh wave travels in an “outer skin” of the material because its penetration depth is approximately equal to its wavelength [18].

Hence using high frequency probes allows to measure elastic properties of the surface layer of the material to a depth of a few millimeters only which is very suitable for studying surface weathering of monuments.

Rayleigh wave was used in stone testing primarily for measurement of elastic constants of stones [19].

This possibility comes from the fact that elastic constants can be expressed by means of longitudinal (compressive) and transversal (shear) wave velocities.

Let us denote the measured velocities of shear and longitudinal waves as V_S and V_P respectively.

Then the isotropic material constants called “dynamic material constants” can be expressed by the equations:

$$\nu_d = \frac{0.5 - (V_S/V_P)^2}{1 - (V_S/V_P)^2}$$

$$E_d = \rho V_P^2 \frac{(1 + \nu_d)(1 - 2\nu_d)}{(1 - \nu_d)}$$

$$G_d = \rho V_S^2$$

where ν_d is the Poisson's ratio, E_d is the Young's modulus, G_d is the shear modulus and ρ is the material density. The above elastic constants can be well estimated when surface wave velocity (Rayleigh wave) V_R multiplied by a factor of 0.9 is used instead of shear wave velocity V_S . This comes from theoretical consideration given in [20] where it is shown that V_S can be approximated by V_R with a correction multiplier depending on Poisson ratio of the material. For stone and brick materials having Poisson ratio close to 0.2 it is justified to use 0.90 to 0.92 correction multiplier (Figure 21).

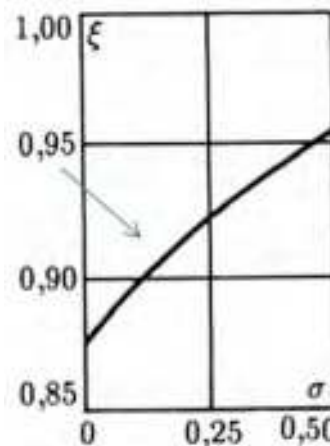


Figure 21. V_R correction factor depends on Poisson ratio.

The edge probes developed by Pęski in 1984 [21] can introduce the surface wave in various kinds of materials without any coupling.

The idea can be described as follows. When a suitable

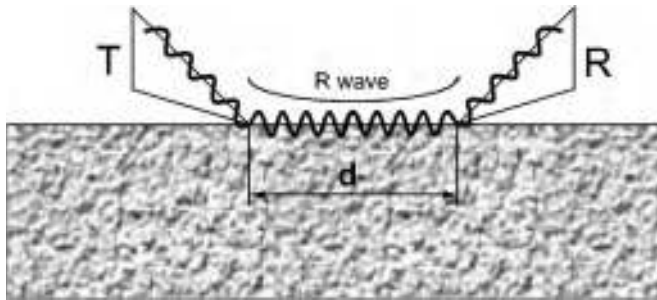
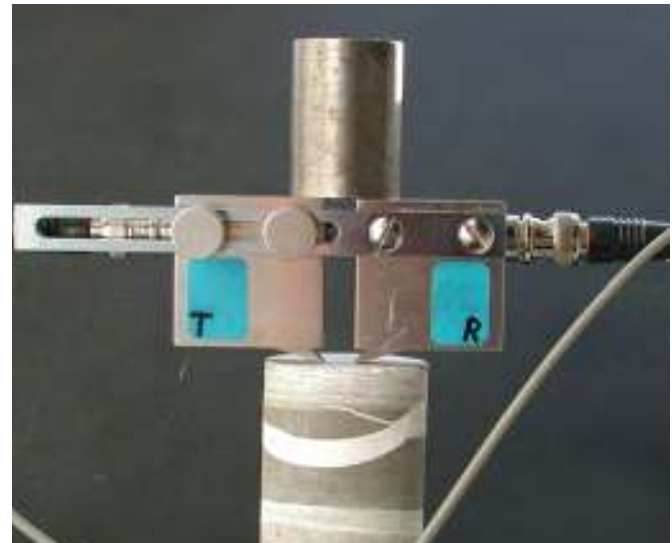


Figure 22. Rayleigh surface wave generation left and measurement right.

inclination of a steel edge transmitter and receiver is used then longitudinal wave emitted by the transmitter refracts at the air/stone boundary and travels in a stone as a surface wave.



After reverse refraction the wave enters the receiver as a longitudinal wave.

Figure 22 illustrates this principle and shows the edge probe during measurement in laboratory conditions. In the Figure 23 two examples of application of surface



Figure 23. In-situ measurements using edge probes: left, St. John's church in Gdańsk (Poland); right, Santa Maria del Fiore in Florence (Italy).

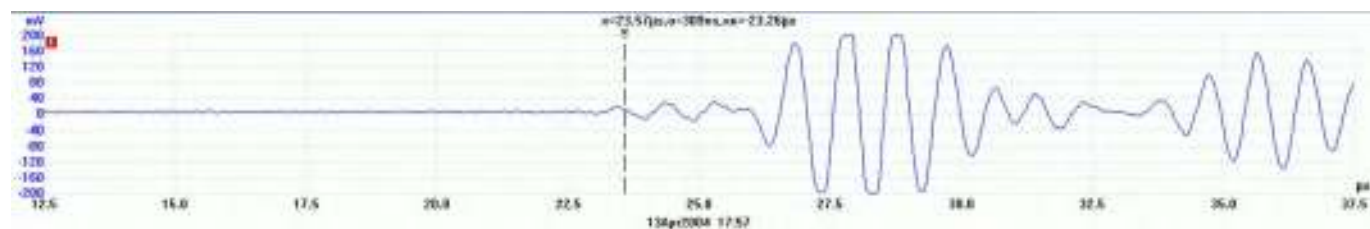


Fig. 24. Multi-wave form of V_R recorded at the south wall of St. John's church.

wave measurements in-situ are presented. One shows V_R measurement on a brick wall of a gothic church in Gdańsk (Poland) and the other one on a marble wall of the cathedral in Florence (Italy).

Figure 24 shows the surface wave registered in one of the bricks of the south wall of St. John's church in Gdańsk. Multi-wave pattern is probably related to an exfoliation of the brick surface, when the surface wave travels in exfoliated and base brick material.

Another interesting example of application of surface wave to diagnostic of adobe wall can be found in [22].

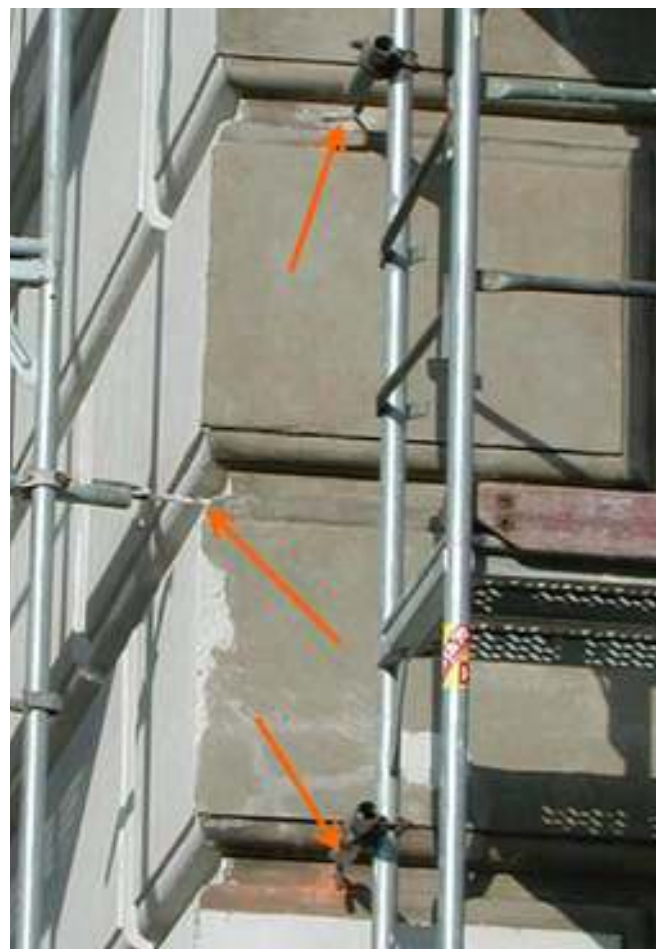
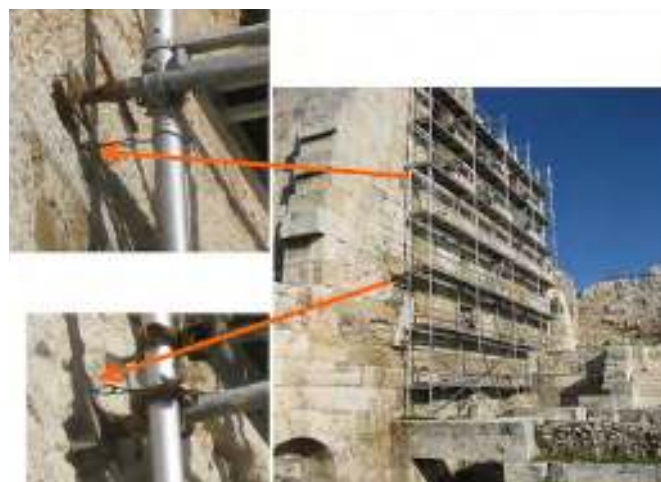


Figure 25. CoDiT is non-destructive from the structure point of view if micro-cores are taken e.g. during scaffolding fixing: left, Athens, Odeon of Herod Atticus, Acropolis; right, Warsaw, Holy Trinity Church.

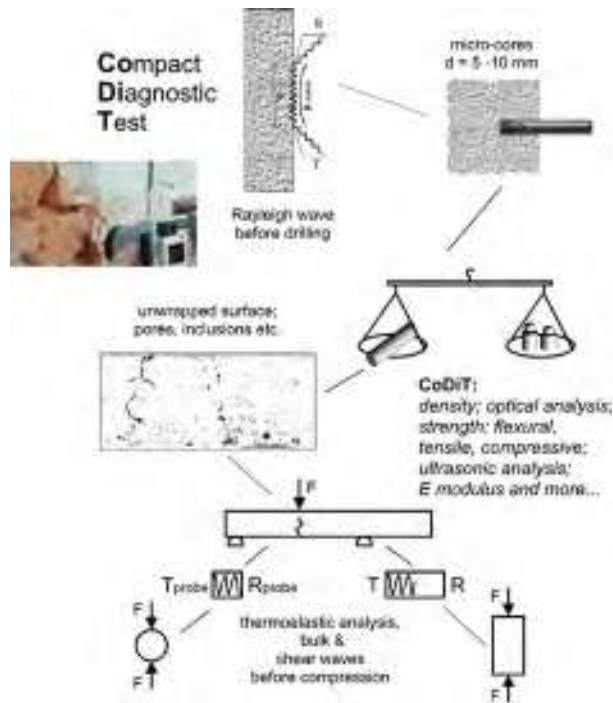


Figure 26. Graphical illustration of Compact Diagnostic Test procedure.

6.2. Compact Diagnostic Test (CoDiT)

The idea comes from the observation that *small holes are drilled in every heritage structure* and that the *valuable historic material is pulverized and lost*. Preliminary information about micro-cores technology was published in



Figure 27. Micro-cores of Schilfsandstein.

IFTR Reports in 2006 [23]. In the same year first results were presented at conferences [24, 25].

The technology is called Compact Diagnostic Test (CoDiT) and is aimed at using sub-sized specimens of heritage building materials either over-cored directly for CoDiT or drilled out during conservation/restoration actions (e.g. scaffolding fixing – Figure 25) or implementation of monitoring sensors or fixing lightning or illumination systems etc.

CoDiT is a sequence of actions allowing us to measure many physical and mechanical properties of heritage construction materials using micro-cores. The sequence is illustrated in Figure 26.

Using a *single micro-core* one can:

1) *Measure*

- 3 ultrasonic waves velocities (surface Rayleigh wave



Figure 28. Sequence of the strength tests using a micro-core: left, FS; middle, UCS; right, ITS.

V_R before drilling);

- modulus of elasticity E during unloading;
- material flexural strength FS , indirect tensile strength ITS , uniaxial compressive strength UCS ;
- material density;

Laboratory & method	IPPT: CoDiT results		BLDF: standard sized	
	Average value	St. dev.	Average value	St. dev.
V_{wave} velocity [m/s]	2700	126	2700	100
E modulus [GPa]	3.8	0.8	4.2	0.8
FS [MPa]	7.2	1.2	7.1 ^{*)}	0.3
UCS [MPa]	46	6.8	50	13
ITS [MPa]	3.8	0.4	1.4-1.7 ^{*)}	

*1) $\Rightarrow$ biaxial flexural strength (discs, diameter 76 mm, thickness 8 mm).

*2) $\Rightarrow$ "direct" tensile strength (TS), the relation ITS/TS is about 2:1 [28].

Table 3: Comparison of the results of CoDiT and other laboratory measurements.

2) Analyse

- statistical validity of the results using Shapiro-Wilk test (elaborated for palaeontology) [26];
- pores, inclusions, micro-cracks;
- chemical composition using spectrometer.

Cross-validation tests on Schilfsandstein specimens were done in 2008 together with Bayerisches Landesamt für Denkmalpflege. BLFD [27]. At first CoDiT measurements were performed on sub-sized specimens (Figure 27) without a knowledge about the material properties (a blind test).

In agreement with the test sequence shown in Figure 26 at first it was performed the measurement of stone flexural strength FS in three-point bending test (Figure



Figure 29 Micro-cores from bricks from South-East part of the Supraśl Monastery (Poland).

28, left) in such a way that the specimen cracked into two various length pieces: longer one used to produce specimen of slenderness 2:1 taken for uniaxial compressive strength UCS measurement (Figure 28, middle) and shorter one, cut for slenderness 1:1, used for indirect tensile strength ITS measurement in Brazilian test (diametral compression) as shown in Figure 28, right. The acceptance of the result is based on visual inspection of the correctness of the observed failure mech-

Measured property	Min	Max	Median	Mean	Std. dev.	No. of spec.	W. val.	p (normality)
V_p [m/s]	2660	2880	2695	2690	515	18	0.9615	0.602
FS [MPa]	4.8	12	2.5	2.6	0.8	4	0.9343	0.820
UCS [MPa]	25.7	37	36.2	34.9	6.3	15	0.9811	0.972
ITS [MPa]	4.8	2.9	3.7	3.6	0.5	15	0.9991	0.889

Table 4: Statistical analysis of measured flexural (FS), compressive (UCS) and tensile (ITS) strengths and longitudinal wave velocity (V_p) for brick specimens.

anisms. They must be the same as known for standard size specimens in each test.

Then the results were sent to BLFD to compare them with standard tests results. Compressive strength

measurements were done using cubes of $5 \times 5 \times 5 \text{ cm}^3$ this means with slenderness ratio equal to 1. Such specimens show the compressive strength values 10-15% higher than the values measured on specimens with the slenderness of 2.

This is in perfect agreement with the results presented in Table 3, where BLFD test gave UCS of 50 MPa and CoDiT test of 46 MPa.

Modulus of elasticity E is calculated in CoDiT using a

slope of the unloading part obtained during UCS loading-unloading measurement sequence.

Tensile strength of the sandstone was 3.8 MPa for CoDiT indirect measurements (ITS) and 1.4-1.7 MPa for direct tensile measurements in BLFD lab.

The CoDiT result is acceptable because “indirect”/”direct” tensile strength is known to be about 2:1 [28].

The comparison of the results of CoDiT and laboratory measurements of BLFD is presented in Table 3.



Figure 30. Displacement monitoring using a smartphone: left, equipment; right, smartphone position on the altar's cornice in front of the canvas.

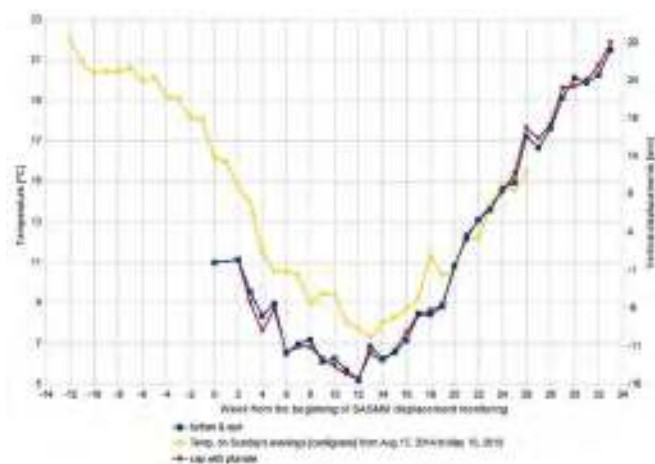


Figure 31. Canvas displacement monitoring results based on smartphone measurements.

6.3. Site application - bricks from Orthodox Monastery in Supraśl, Poland

CoDiT measurements were done to evaluate a vertical profile of possible masonry brick degradation in a lower part of the Monastery building [29]. The assumption was that brick layers close to the ground level can be more weathered due to water action as snow and rain are often in the area.

After performing measurements of V_p , FS, UCS, ITS a conclusion was that no meaningful differences in brick properties were recorded and hence that weathering of the lower part of the wall was not pronounced.

The conclusion was based on a statistical analysis of the measured brick properties. To analyse whether measured results come from populations having normal distribution Shapiro-Wilk normality test (W statistics) was used [26].

The choice of this test is justified by a small number

of samples in each result group.

W statistics given in Tab. 4 is accompanied by $p(normality)$ value, calculated for the Shapiro-Wilk test by PAST software [30] used to confirm/reject assumption that the measured value comes from the normally distributed population. The compressive strength UCS is very likely normally distributed, despite the standard deviation as large as 42% of the mean value. In contrast the tensile strength ITS measurement almost certainly deviates from the normality with $p(normality)$ of 0.08. The other two material parameters, the flexural strength FS and longitudinal wave velocity V_p , are probably normally distributed but with low probability of around 0.6 only. These results indicate that the minimum, maximum and median values of the measured parameters might be more informative for practical purposes than the mean value and standard deviation.

6.4. Smartphone aided Structural Monitoring and Measurements

Smartphone Aided Structural Monitoring and Measurements (SASMM) [31] was presented at the SAHC 2016 Conference [32] where four applications were described:

- marble sculpture photogrammetric documentation,
- stone dampness evaluation,
- measurement of horizontal vibrations induced by a town bus stopping and
- monitoring of deformations of a large canvas painting.

The use of a smartphone for a long time deformation monitoring was probably done for the first time ever and SASMM concept adopted to canvas monitoring was successful. The canvas and its documentation is presented above in Example 4 of the section “Documenting Examples”.

After conservation and restoration works the canvas was repositioned on the church apse wall and monitoring equipment consisting of the smartphone, an external additional battery with a small solar panel and a standard socket charger was used (Figure 30, left). The monitoring equipment was positioned on the altar's cornice with the smartphone camera facing the canvas (Figure 30, right). Several images were taken immediately after the smartphone positioning to test various illumination conditions and their usefulness for monitoring purpose. Taking the monitoring images during the evening church service when most of the light would be switched on independent of the year period was decided.

The canvas was photographed once a week during a period of eight months and displacements were calculated using the image correlation of several areas (masks) naturally existing in the painting. No additional marks were used.

The procedure was proposed in year 2006 for a historical fresco monitoring and described in [16].

Locally measured canvas painting displacements during the monitoring period are presented in Figure 31. They showed no deformation gradient between the monitored masks positions and hence no creep strains manifested during the monitoring.

Only global displacements were measured and they can be attributed to thermal movements of the canvas and the structure.

The painting is displaced in Namur Cathedral, Belgium, and the whole displacement monitoring was supervised via Internet from Warsaw, Poland.

The images were taken automatically driven by a software inside the smartphone and automatically sent to

a server. The correlation analysis was done off-line in Poland using FreeMat software.

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Abstract

The study of historical constructions oriented to their preservation requires multidisciplinary teams of specialists formed in relation to the type and the scale of the problem. The design of interventions must be based on scientific understanding of the historical construction accurately. The study of building construction method and structural details has a great importance in the mechanical and dynamic behaviour of a building.

Masonry structures are constructed of small units as fired or sun- dried bricks, concrete or stone units. The resistance depends on; the geometry of the structure, the strength and stiffness of the materials used, and how the masonry units are joined together. Main components of masonry structures constitute foundations, columns and piers, walls, floors and roofs.

This paper will present the construction method of historical masonry components so that their behaviour under different actions can be understood better. To make sound intervention decisions; it is essential to know the technological properties of the masonry construction.

Keywords: masonry walls, opening in walls, vaults, domes, foundation

1. Introduction

The preservation problem of historic structures requires: documentation (inventory of the building through historical and architectural survey), diagnosis (identify causes of damage and decay), safety evaluation (analysing present condition of structure and material), decisions on intervention (selection of efficient repair or strengthening methods to ascertain the required safety) and monitoring, (control of the efficiency of the interventions). These studies should be carried out with a multi-disciplinary team of specialists formed in relation to the type and the scale of the problem.

Most of the architectural heritage buildings survived up today are of fired or sun- dried bricks, concrete or stone units. Brick and adobe are clay or shale masonry units, fired or sun dried. Bricks are less absorptive than adobe units. Units are solid or cored. To form a structural element, masonry units are laid dry, joined to each other with metal clamps or laid in mortar. Masonry units have high compressive and low tensile strengths. Masonry structures withstand lateral loads with the movement of their units [1]. In dry joined masonry, compressive strength of the masonry unit



Figure 1. Flexible foundation base.

and friction between the units resist tensile forces. To enable the movement of the units, metal connectors were laid in lead; mortar had less strength than the masonry unit. Using weak, rather than strong mortar let sliding along the bed joints dissipating energy so that the masonry unit will be safe. Ignorance of this knowledge and construction technique may lead to incorrect interventions.

2. Foundations

The foundation construction of the walls depended on the type of soil. Soft soil needed to be consolidated by wooden piles. The construction of foundation wall was either solid or cavity wall constructed of two rows of stones, where the space between the rows is filled with the rubble and earth. During the process of time as the wall thicknesses became thinner, the smaller sizes of foundation walls widening at the foot were practiced. If the wall was composed of closely spaced piers and there was not enough



depth to step the foundations, sometimes inverted arches or vaults were made [2].

In earthquake areas; adequate construction would be stiff structure on soft soils, flexible structure on hard soils and rock to prevent resonance. At high seismic zones, the lateral stability of structures was provided by changing the natural frequency of soil or providing flexible base (Figure 1). If the rock bed or hard soil was deep, the foundation walls rested on a pillow layer of sand or gravel; If the rock bed was close to the surface, a flexible base was provided by placing single or more layers of wood at the foundation base [3]. If the rock bed was on the surface, the rock was carved in the form of trough so that each stone of the foundation wall could be placed in the rock as if resting in a cradle [4]. In between the rock and the foundation stone, briar, pieces of coal or animal skin was placed [5].

3. Masonry columns and piers

Columns are composed of one unit of stone in



Figures 2a, 2b. Columns and Pier.

cross-section (Figure 2a). A column can be of one piece or several pieces placed one over another. In connection of columns to each other and to other elements are made either thick units rest dry laid one over another or thin units with metal connectors laid in lead. Piers are masonry wall segments constructed of small masonry units (Figure 2b).

4. Masonry walls

A thick masonry wall may be of solid wall, cavity wall leaving the space between the outer faces empty, cavity wall filling the space between the outer faces with rubble and mortar or, lime concrete veneered with masonry units (Figure 3).

The outer leaves of the cavity walls, with or without infill, are joined with suitable lacing (tie bands) at certain intervals. Lacings of timber, brick, stone or metal ties limit the support movements, prevents wall bulging under gravity loads, reduces the slenderness ratio of the wall and prevents crack initiation at another location. Iron or copper ties were used after 1840. These ties were joined to each other either with metal agrafe, seen from the outside of the wall or inserted in lead in a round clevis, not apparent from outside. It is important to identify their place in order to assess the slenderness ratio of the wall. Slenderness ratio of a wall is very important in resisting earthquake forces and diaphragm requirement of the floor or roof.

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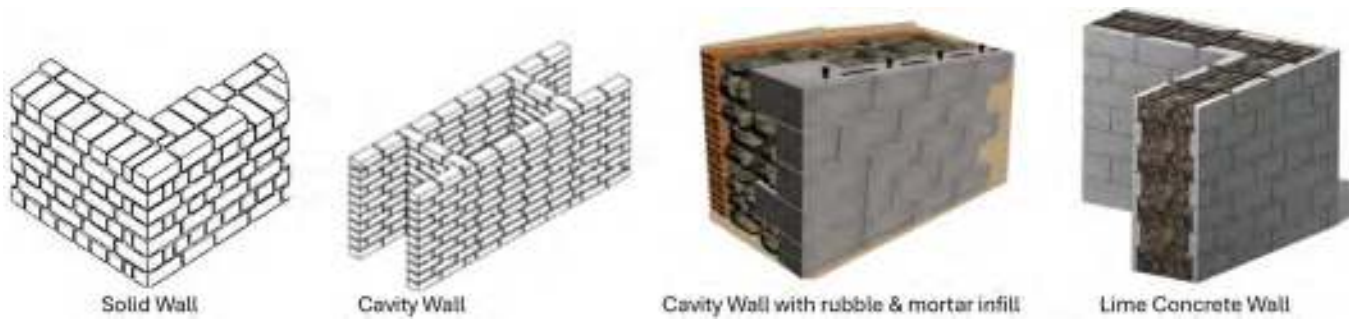


Figure 3. Wall Types.



Figure 4a. Openings on the walls: arch, corbelled arch.

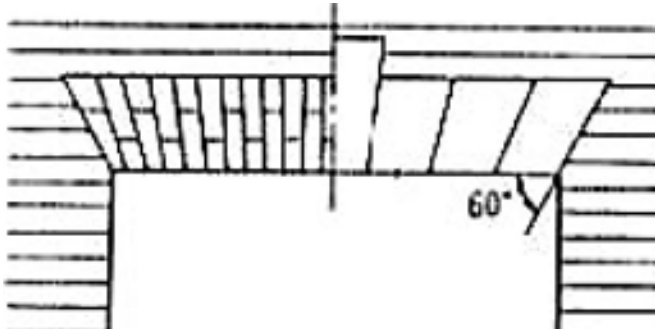


Figure 4b. Openings on the walls: flat arch.



Figure 4c. Openings on the walls: lintel.

In historic masonry construction, the openings for windows and doors were spanned by lintels, by flat arch, by corbelling or by arches (Figures 4a, 4b, 4c). *Lintels* are horizontal elements of stone blocks, iron profiles or timber that span the door or window openings. Length of the span depends upon the material's modulus of elasticity. The *flat* or *lintel* arch, is made of small masonry units with a horizontal soffit

(instead of curved), the ending at an angle of about $\alpha=60^\circ$. Curved arches are constructed bedding the stone or brick units horizontally or radial, perpendicular to the profile curve. Arches made up of horizontally laid units extending into the opening are called corbelled or false arches. Corbelled arches act as a wall and the arch action is eliminated. The arches with units laid radial to the profile curve are called arch or true arch. All loads acting on the arch are carried sideways, as well as downward by means of inclined thrusts passing through the arch units. To take this outward thrust on their supports; an enough mass of wall or another arch with opposing thrust at the supports or an effective tie between the supports are constructed.

5. Floors and Roofs

Floors and roof on masonry walls can be framed with wooden beams, joists and rafters, timber or steel jack arch, lintel arches or vaults and domes.

The configuration of the units as corbelling or radially laid units in forming the vaults and domes is very important in the component behaviour. If the vault and dome configuration is corbelled of horizontally laid units, all the loads acting on the curved surface is carried vertically from unit to unit downward. Behaviour is like a masonry wall.

If the configuration is radially laid units, units laid perpendicular to the profile curvature, identification of the geometry of the surface is very important to understand the distribution of forces and predict possible weaknesses. This time, the surface acts as a doubly curved surface, having two principle stresses. Vaults with radially laid units may be cylindrical,

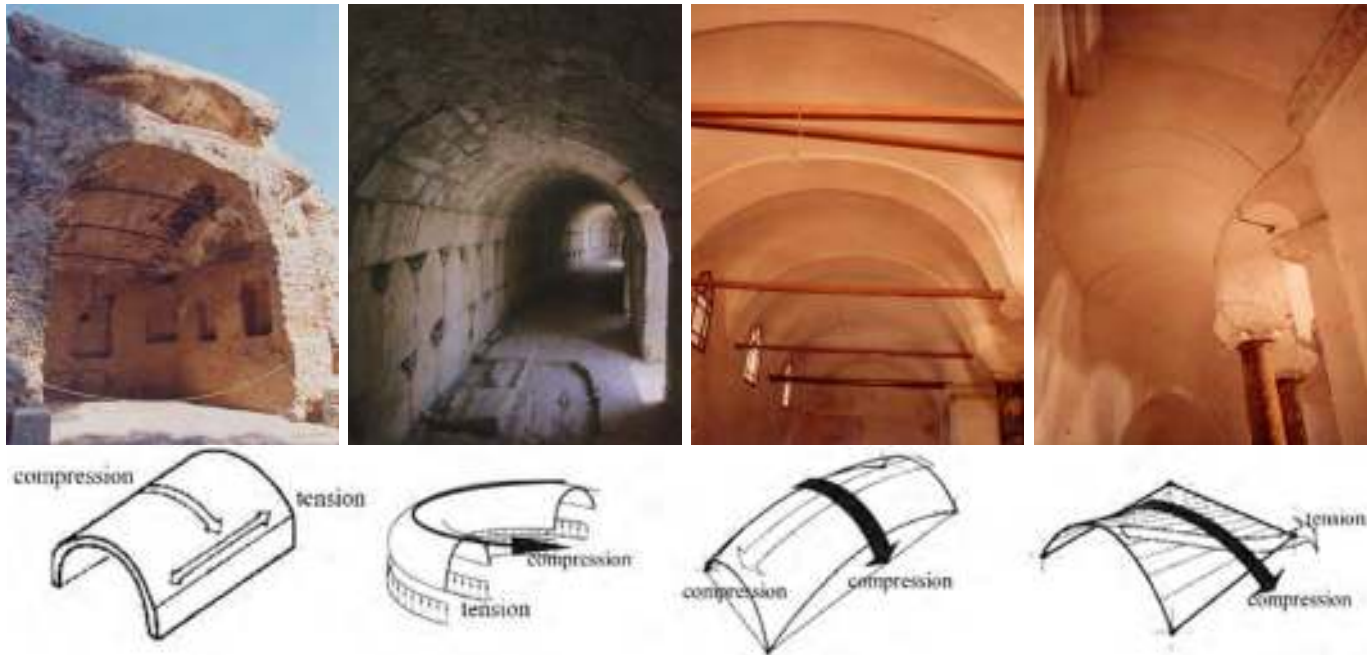


Figure 5. Vault geometries and their surface stresses.

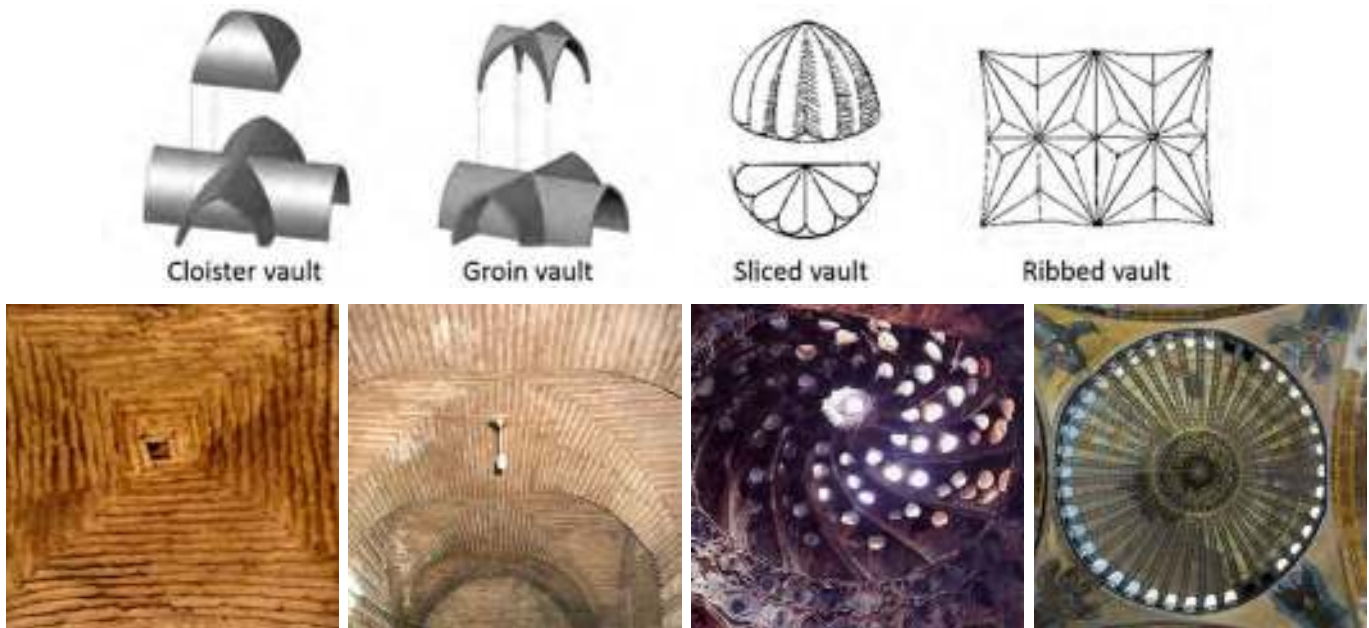


Figure 6. Domes made up of vaults.

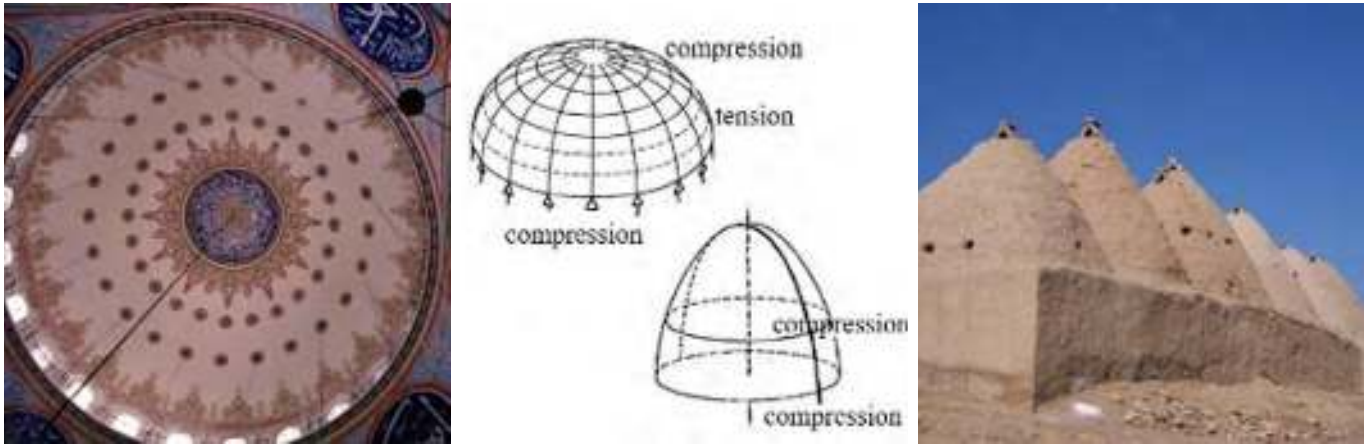


Figure 7. Rotational Domes.

conical, elliptic paraboloid, torus or conoid. Some vault geometries such as cylindrical and torus vaults inherently have tension at the base along one of their principal curves (Figure 5).

To span square spaces, by organization and intersection of these vaults, domical vaults as; cloister vault, groin vault, sliced vault or ribbed vault are obtained (Figure 6).

Rotational domes are derived by rotating a meridian curve around a fixed vertical axis. Variety of forms of domes can be obtained by rotating a circular arch, a half-ellipse, a parabolic or any other kind of a curve around the vertical axis. Domes with radially laid units are doubly curved surfaces that have one set of principal stresses radially from crown to base, and another set circumferentially (Figure 7).

Spherical domes inherently have tension at the base of circumferential curve.

To exterminate the tensile stresses in the tensile area of the vault or dome surface with radially laid units; this area is thickened with heavy material or closely

spaced arched columns or vaults perpendicular to the surface is provided or the tension part is changed to corbelled system (Figure 8).

Usually, round based domes are organized over a rectangular plan. To cover the corners of such plan, transition elements as pendentives, corbelled triangles, squinches and corbelled triangle bands are used (Figure 9).

6. Structural organization

6.1. Seismic Joints

In the building geometry, long projecting and different weighing components of a building were separated as expansion or seismic joints. Here, the building parts staying side by side were separated by not joining the masonry units to each other (Figure 10).

In many cases of archaeological excavations, it is seen that new structures have been built up-on old foundations. In this case, the inner grillage foundation walls were not completely bounded to the perimeter wall foundation.



Figure 8. Extermination of tension area at the surface.



Figure 9. Transition elements.

When different components of a building were not of the same weight and would have settled differently, the intersecting lines of the foundations were not bounded.

6.2. Underground Water Control

Water is the most serious non-seismic threat to masonry buildings in areas of both high and low

seismicity. Depending on the porosity of foundation construction material and soil characteristics, it can damage the wall and soil by actually eroding away portions and by reducing the strength.

So underground water drainage was given high priority during the design and construction of historic structures.

In Anatolia, the major components of the drainage



Figure 10. Seismic joints in Süleymaniye Mosque and Çifte Hamman.

system to drain interior ground surfaces of a building included wells or cistern in the basement, galleries or channels that discharge the water out of the building, and gates out of the building, usually in the garden for ventilation. This system removed the moisture from underground and surface water. If this system is broken, water can damage the wall and soil by eroding away portions and by reducing the strength.

7. Final remarks

Ancient engineering systems are different than current practice ones. The knowledge of the old construction technique was lost during the last century. In investigating heritage buildings, it is important to consider the engineering knowledge of their construction time and the construction method of structural components.

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Geotechnical Engineering for Conservation of Heritage Structure

A case study to identify the mechanical reason of NI Tower, Prasat Suor Prat in Angkor

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Abstract

Two case studies are described in terms of sustainable conservation of cultural heritage as well as social development in the World Heritage Site of Angkor, Cambodia. These problems are very unique in a sense that the phenomena were found quite different from what geotechnical engineers had been used to expect in daily practice. A tower of masonry structure that JSA (Japanese Government Team for Safeguarding Angkor) selected for structural conservation was inclined about 5 degrees. Since the deformation of the step stones showed the same inclination, tilting of the foundation was considered as major mechanism for the inclination of the Tower. After trenching of the soil mound, the compacted soil layer was not inclined but horizontal. This was quite different what we had expected. What was the mechanism to cause the inclination? Further study revealed that the slip-down of the side step stones along the foundation mound was the true mechanism. JICA (Japan International Cooperation Agency) made a study of water resource from pumping underground water. They have monitored seasonal fluctuation of shallow water level at more than 50 surface wells and tried to simulate the seasonal changes caused by horizontal flow from higher mountain to lower lake

zones. Finally, JICA had installed a facility to pump water more than 8,000ton/day. After the pumping started, a monitoring well at 3km from pumping zone showed 1m of drawdown of water level that was predicted as little effect by the simulation. Since the plain in Angkor has very gentle slope of the order of 0.001, horizontal flow of underground water is very slow and negligible. The seasonal changes of the water level of 4-5m was caused not by horizontal flow but infiltration of rain water and evaporation water from the deep layers that was not included in the model JICA used.

Keywords: Conference, Analysis, Construction

1. Introduction

When the turmoil in Cambodia has ended, Japanese Government had an inter-governmental conference on Safeguarding Angkor in October, 1993, Tokyo I). The Japanese Government Team for Safeguarding Angkor (JSA) was formed in 1994 and the author had joined as geotechnical member. We have been engaged in studying geotechnical problems in Angkor, Cambodia since then. JICA (Japanese International Cooperation Agency) worked from 1996 to 2000 to perform a



Figure 1. Map of Cambodia.



Figure 2. Map of Angkor.

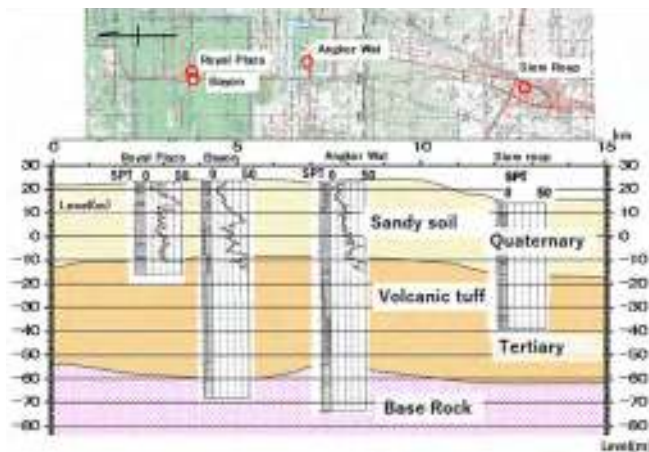


Figure 3. Borings and Geological NS Section in Angkor.

feasibility study of possible water resources in the Angkor area for citizens in Siem Reap city. Based upon the study, a pumping station was constructed at the south of the West Barai and started its operation in 2006.

2. Geotechnical Setting of Angkor Region

Angkor locates at a wide plain that expands between the Kulen Mountain in the north and the Ton le Sap lake in the south as shown in Figures 1 and 2.

The change of the ground height at the lake to the foot of the mountain is about 50 m with a distance of 50 km. The slope of the Angkor plain is very gentle of 1:1000, in which Siem Reap River, originating from the Mt. Kulen runs southwards to the lake. The eastern Kulen shows a special topology of bowl type shape and is a water reservoir during rainy season. The Siem Reap River flows continuously even in the end of dry season after no rainy of 6 months.

Geology of Mt. Kulen is composed of sandstones that

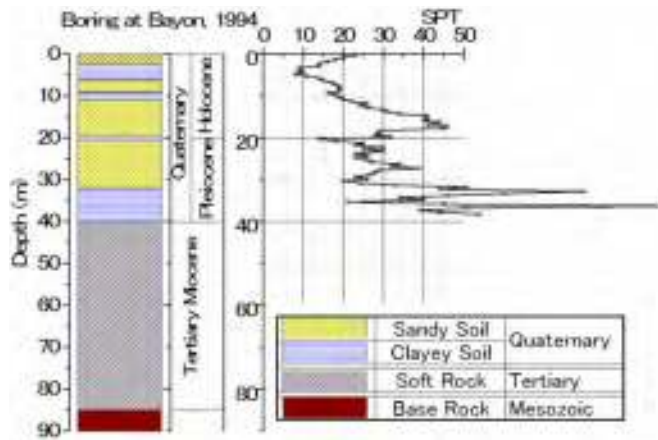


Figure 4. Geological boring at Bayon.

had been used for construction material for masonry structures in Angkor.

JSA performed several long drillings of 100m to reach the bedrock geology. The geological layers found by the boring study are sediments of Quaternary, Tertiary, and base rock. Base rock is an igneous rock of andesite. The results at the center in Angkor region are shown in Figure 3. The base rock in the Angkor region consists from igneous rock to sandstone in the Mesozoic era. Figure 4 shows one of the boring logs at the north of Bayon temple. The upper 40 m is Quaternary deposits of silty fine sand with several clayey rich layers. In general SPT (standard penetration Test) N-value increases with depth except at the top surface. N-value at the top surface is 20 and decreases to 10 at the depth of 5 m. The boring was carried out in March 1995 of dry season. The cone penetration test that was performed in rainy season at the same site revealed that the cone resistance decreased to zero at the surface and increased with depth down to about 5 m. The top surface of 5 m may be affected by the seasonal

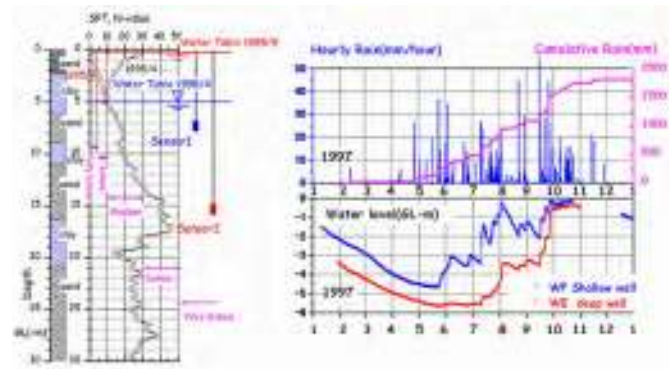


Figure 5. Rainfall and seasonal change of pore water pressure at Bayon site.

change of water level. Pore pressure sensors were installed at two depths as shown in Figure 5. The pore water increases during rainy season from May to October and decreases in dry season from November to April. The fluctuation of the water level was 4 to 5 meters.

3. Tower of Prasat Suor Prat⁽³⁾

Prasat Suor Prat (Tower of Rope Dancer) is a group of 12 independent towers in the Royal Plaza, Angkor Thom as shown in Figure 6.

One of the 12 towers named as N1 that is inclined about 5 degrees northwards was selected by JSA for safeguarding work. The tower consists from a main tower with a front room called antechamber. The main tower is three story masonry structures with laterite bricks with 10 m in width at foundation and about 20 m in height. The inside of the tower is hollow structure with four open windows at every wall.

Figure 8 shows vertical section and plan view of the N1 tower (Figure 7) near the north pond. The differential



Figure 6. Prasat Sour Prat (Tower of Rope Dancer).

settlement of the foundation was measured about 40 cm at the north and west corner lower than that of the south and east corner. Figure 8 also shows contour lines of equi-settlement relative to the south-east corner of the foundation.

The open window of the west wall is deformed as shown in Figure 9. The widths of the bottom of the window are wider than those of the top for all four walls and the measured results are shown in Table I. The laterite blocks of the side face of the foundation are found widened. The characteristics of the deformation of the foundation are settlement and horizontal spreading. What kinds of deformation could result in the measured characteristics of settlement and horizontal spreading?

3.1. Expected Mechanisms of spreading with inclination

Possible modes of deformation are shown in Figure 10. They are as follows,

A. Uni-axial compression with lateral expansion with



Figure 7. Prasat Sour Prat Tower N1.

differential settlement,

B. Base failure with bi-lateral failure, or

C. Base failure with unilateral sliding.

Case A may be under a rather confined plastic state with differential compression, Case B and C are under yielded compression with sliding state. Since the horizontal expansion at the foundation is 6-8%, the axial compression strain may have reached more than 10%, the deformation are very likely under failure and B or C was considered more appropriate than A (Table I).

3.2. Trenching of the foundation

Dismantling of N1 Tower of Prasat Sour Prat was carried out before reconstruction of the Tower for

Position	Width of Opening		Difference	Expansion
	Top	Bottom	ΔL	(%)
	Lt(cm)	Lb(cm)	Lb-Lt(cm)	
East	170.3	182.0	11.7	7.1
West	171.3	185.0	14.7	8.2
North	168.0	178.0	10.0	6.0
South	124.0	131.7	7.7	6.2

Table I. Width of open window at top and bottom.

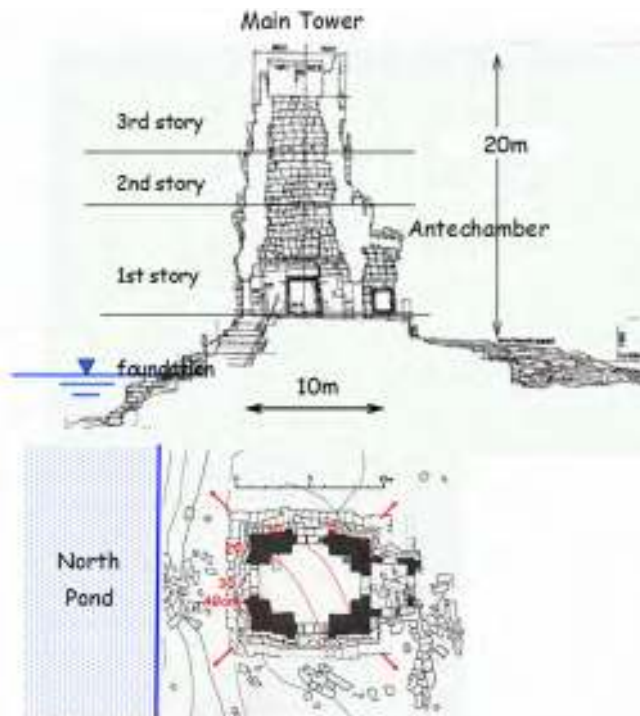


Figure 8. Vertical and plan sections for NI Tower.

safeguarding the structure. After dismantling laterite blocks of the upper structure, the soil mound was further excavated down to reach natural ground layer. The archaeological team of JSA excavated several



Figure 9. Open window of the west side of NI Tower.

sections as shown in Figure 4. Sketch of the NS-section along the center line is shown in Figure 8. To our surprise, each layer of the compacted soil mound was found horizontal. If the scenario of the deformation was as shown in Figure 7, in any cases, the soil layer must have been inclined to northwards. The excavated

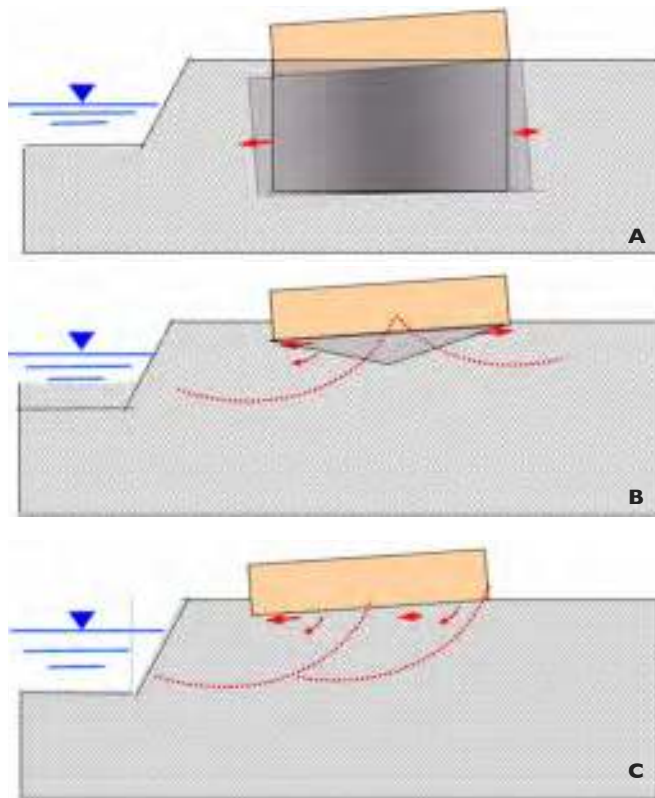


Figure 10. Possible three geotechnical models for the inclined foundation.
 A. Uni-axial compression with lateral expansion with differential settlement.
 B. Base failure with bi-lateral sliding.
 C. Base failure with unilateral sliding.

horizontal layers were out of our scope before dismantling. However, soon later, we found interesting facts and could get out of the pitfall.

3.3. Streak lines on the excavated trench surface

Figure 11 shows a typical trenched section that revealed many streaks in whitish color. The widths of the lines of the streaks are about 5mm and somewhat

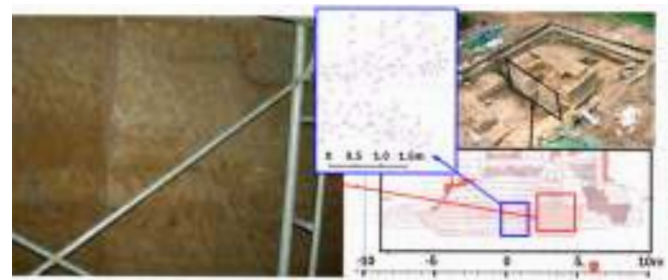


Figure 11. Streak pattern found on the trench section.

weaker than the surrounding mound surface. These streak patterns of the area are analyzed. The distribution of the angles of the streaks from the vertical line were analyzed. Typical results are shown in right part in Figure 12. It turned out that two angles of +30 and -30 degrees from the vertical plane are predominant in the pattern.

$$\theta = 45 \pm \phi/2 \quad (1)$$

The laboratory triaxial test for unsaturated sample showed that the internal friction showed is:

$$\phi = 30 \text{ degree, which result in } \theta = 30 \text{ and } 60 \text{ degrees.}$$

Figure 12 shows Streaks on the trenched face and the distribution of the angle from the vertical line under the Mohr-Coulomb criteria, the angle between the direction of failure surface and principal stress line is expressed by the following equation (1) with ϕ as the angle of internal friction of the material. The area is divided into upper and lower parts and two charts correspond respectively.

The coincidence between the estimated angle of shear failure line from the laboratory test and field observation, the streak lines is very likely the shear failure line

that have occurred in the soil mound under the vertical load of the upper masonry structures.

3.4. FEM Simulation of the stress in the soil mound⁴⁾

FEM simulation was carried out to check the stress state in the soil mound of the foundation. The Model is shown in Figure 13.

The FEM analysis was performed assuming the Mohr-Coulomb criteria for yielding with parameters as listed in Table-2.

The results were obtained for two cases of loads of 100kPa and 400kPa upon the foundation mound from the upper structures.

Figure 14 shows distribution of plastic points in the soil mound induced by these steps. The 400kPa is the expected load from the upper structure.

The result was found the shear yielding took place at the initial stage of the loading and the yielding points are spread widely in the mound section.

The yielding points become to include the upper region of the mound when the load is increased from 100 to 400kPa. Simulation results, though preliminary, show the reasonable stress states compared to the yielded condition that is estimated by streak lines.

3.5. Slip down of step stones surrounding foundation mound

The results of the archaeological excavation is shown in Figure 15 with photo.

In contrast of the horizontal layer of the compacted mound, the laterite blocks of the step stones that had been installed to cover side slope of the mound were found slipped down along the northern side slope of the mound. It is also found that the northern mound layer beneath the bottom of the side stone block is lower than the continuous layer of the left part.

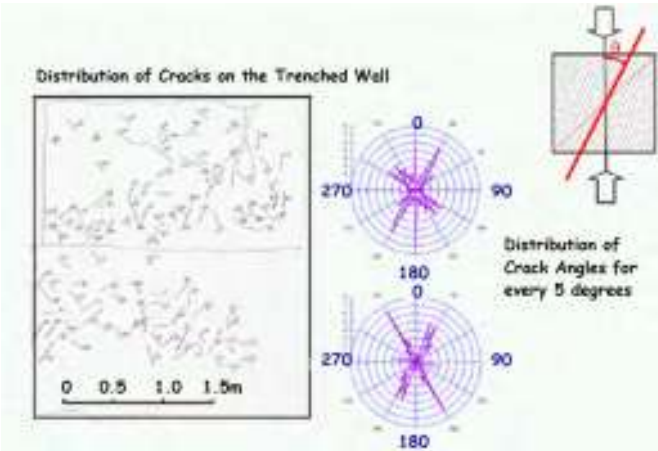


Figure 12. Streaks on the trenched face and the distribution of the angle from the vertical line.

Material	Young's Modulus	Poisson's Ratio	Cohesion	Frictional Angle	Unit Mass
	E(kN/m ²)	ν	C(kN/m ²)	(degree)	(kN/m ³)
Compacted Fill	25,000	0.3	25	30	18
Natural ground	10,000	0.3	25	30	18
Laterite	1,000,000	0.2	0.2		30

Table 2. Material parameter for Prasat Sour Prat.

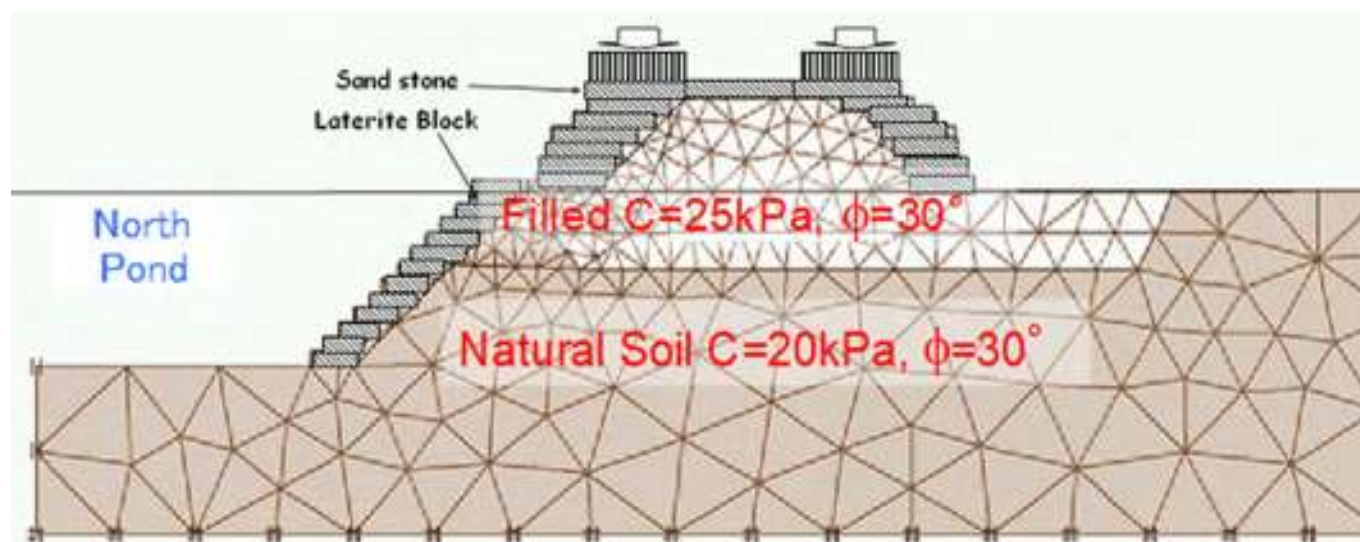


Figure 13. FEM meshed model.

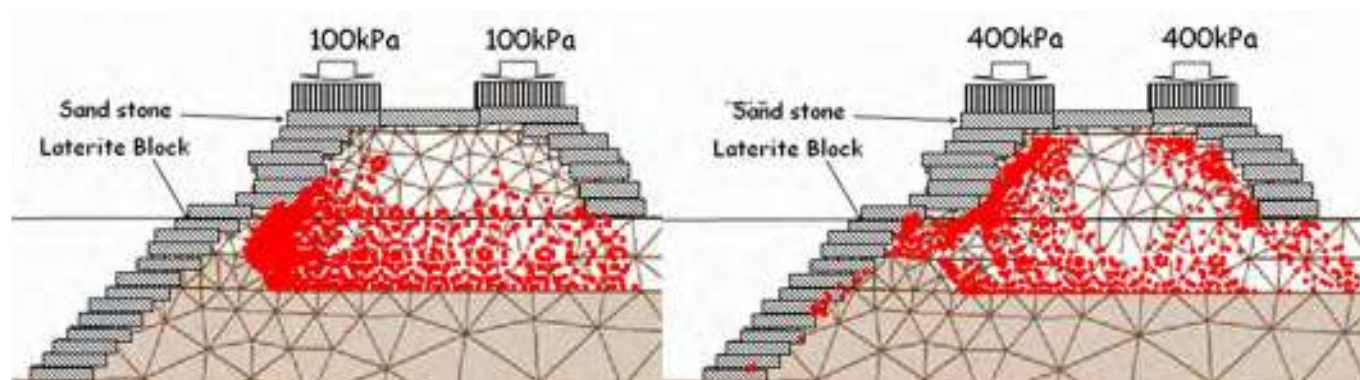


Figure 14. Distribution of yielded points caused by loading from upper structures.

This deformation is considered as to have been caused by foundation failure that resulted in the slip down of the side blocks of laterite.

3.6. Mechanism of the spreading and inclination of the foundation

It was revealed that the several presumed mechanisms

to explain lateral spreading and inclination of the foundation were wrong.

The real situation was the slip down of the side step block along the northern slope of the foundation.

The geotechnical mechanism for lateral spreading was large horizontal expansion associated with vertical compression in the yielded state.

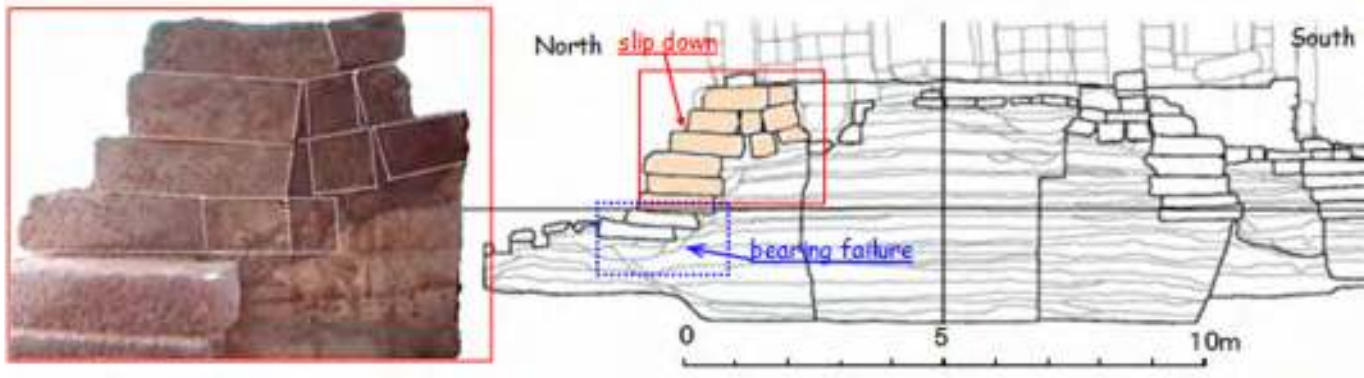


Figure 15. Deformed and slipped down step stone along the northern slope of the mound.

The mechanism for the inclination was not caused by differential settlement of the ground but the slipped down of the side stone along the foundation slope. We had fallen in a pitfall of geotechnical engineering. Without the excavation, we never get out the hole and arrived at the correct mechanism

4. Conclusions

The inclination of NI Tower of the Prasat Sour Prat was really out of the conventional concept of geotechnical engineering. The true mechanism had become apparent in the process of exposure of the trench section of the soil mound. In geotechnical engineering, speculation, or assumed theoretical ideas are very important to grasp the physical phenomena and to understand the problems.

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Abstract

Before deciding on structural intervention, it is indispensable to determine first the causes of damage and decay, and then to evaluate the safety level of the structure with experts of multi-disciplinary team. Every structure is under external loads of nature or man-made actions. These loads acting vertical, horizontal or at an angle result with tensile, compressive, bending, shear or torsional stresses. Today, the historic masonry structures are under loads that were not present during the time of their construction. In this paper, behaviour of masonry components and structures to external loads will be explained.

Keywords: Actions acting on structures, gravity loads, environmental actions, earthquake actions to masonry structure

1. Introduction

Masonry structures are constructed of stone, brick, adobe or concrete units. These units are weak in tension and relatively strong in compression. Their mechanical, physical and chemical properties depend on where the material is taken and how the

units are produced. As every structure, masonry structures are also under external forces imposed by gravity loads, environmental actions as moisture and floods, natural forces as hurricanes and earthquakes and differential settlement of foundation and wrong intervention. These forces exert internal stresses on the masonry components. Due to the high ratio between specific weight to load carrying capacity, masonry structures bear loads in massive and rigid forms. Failures in masonry construction appear as crack and material loss. Both defects indicate the presence of possible weak zones. Cracks appear when the imposed loads exceed the tensile capacity of masonry. Initially the cracks start perpendicular to the tension lines, then progress slightly different directions. Masonry structures are redundant structures. Despite cracking, the structure may keep on resisting safely provided that it can develop a resisting scheme involving compression forces as an embedded arch or system of arches. Cracking in masonry does not necessarily point to structural problems. But such failures on the structure, affect the load bearing capacity and durability of masonry components.



Figures 1a. Vertical cracks; 1b. Loss of the material.



Figure 2. Longitudinal cracks along the arch.

2. Gravity loads

Under compression of gravity loads, close to the ultimate compressive strength of the material, first vertical cracks (Figure 1a), then crushing and out-of-plane displacement may be observed on masonry walls and piers. Cracking and splitting of unconfined parts (as corners), may occur for moderate compressive stresses

below 60% of compression strength [1]. High compressive stresses may cause splitting, and consequent loss of the material (Figure 1b). In rubble filled walls, the inner core might exert thrust to the outer leaves under gravity loads. Under compression close to the ultimate compressive strength of the material, first vertical cracks, then out-of-plane displacement may be observed. Out-of-plane displacement of a wall may cause overall collapse of the building. Failure in compression may occur in the long term, even at moderate average compression stresses, due to the decay of the material or due to long-term damage (creep).

In arches, continued gravity loads may give rise to longitudinal cracks running in the direction of the span near the edge of the arch, to bulging of the spandrels and to cracking of the wing walls [2]. These cracks may indicate that the arch has broken up into narrower rings, each spanning independently (Figure 2). If the cracks are compatible with equilibrium and if the separated parts can resist the load independently these cracks may not



Figures 3. Effect of moisture + wind.

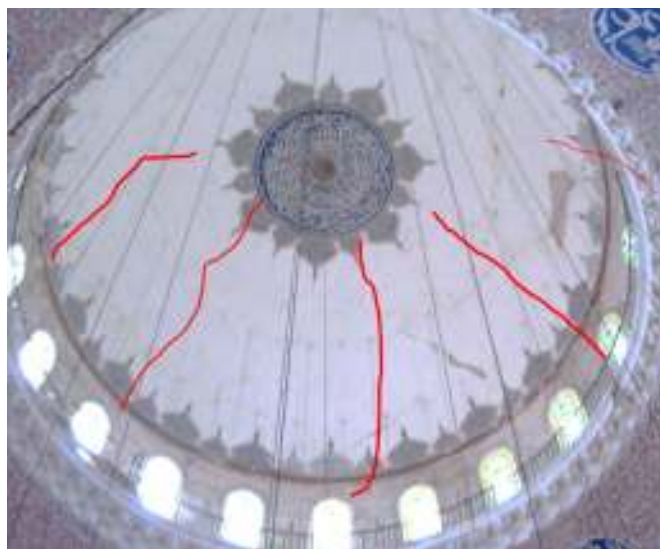
be unsafe. Cracks in the spandrels near the quarter points of the span indicate that the arch is acting as a hinged frame. This also does not indicate a dangerous state unless the fourth hinge doesn't develop. Continuing gravity loads on masonry walls with openings, may foster the generation of relieving arches embedded within the surface. Several unloaded openings below the relieving arch, may crack, loosen and even fall down, but the structure may still be safe. The vaults that have tensile stresses in their geometry, if the thickening of the skirts loosen, tensile stresses become active and exertion of thrust to the supports develop longitudinal cracks. The maximum compressive forces on the diagonal arches of a groined vaults occurring at the springing of the arch might form cracks parallel to the groins. On the dome with radially laid units, excessive gravity loads may cause radial cracks. If the dome crack is as an embedded arch or system of arches, the structure may keep on resisting safely.

3. Environmental loads

Environmental actions may be failures due to flooding, vegetation, biological actions as termites or spiders or air pollution and traffic vibration.

Flood occurs when rivers or streams overflow their banks or the sea drives inland due to excessive rain, a ruptured dam or rapid ice melting in the mountains. The moving water has destructive power on structures poorly constructed. Depending on the porosity of the masonry units, masonry can suffer moisture. The erosive force of flood can affect the building's foundation, causing it to crack.

Moisture caused by rain, surface or underground water can cause leaching of the weak masonry units and discoloration and weathering on masonry components. Absorption and pore structure of masonry material affect their resistance to frost damage. Soft masonry units weakened by moisture may disintegrate and even dissolve with the effect of wind (Figure 3).



Figures 4. Crack on domes left: radially laid units, right: horizontally laid units (corbelled).

Biological actions may be counted as colonization like algae, lichens, mosses, plants, termites and spiders, efflorescence, spalling and fragmentation on the masonry components.

The traffic near the masonry buildings may cause air pollution that leads to spalling and foliation of the masonry units. Vibrations caused by traffic may weaken the material, loosen the connections.

4. Earthquake Loads

Earthquake forces may develop shear, bending and torsional failures. Horizontal forces to columns may cause rocking of the column and relative rotation and sliding of the units that make up the column. High friction between the thick units prevents sliding.

Load bearing walls may resist gravity and lateral loads from overlying floors or roof. Typical damage to masonry walls during an earthquake is in-plane shear

cracking, out-of-plane overturning of walls and separation of adjoining walls. During earthquake, the joints that transmit tension become progressively looser. As cracks appear, friction between the units limits the displacements. Using weak, rather than strong mortar lets sliding along the bed joints dissipating energy. As cracks develop, the dynamic response characteristics change; the fundamental frequency decreases.

On in-plane walls as cracks intersect each other, independent irregularly shaped wall segments form on the surface, and cracks become substantial. In thin walls, such wall segments formed by diagonal cracks may slip and fall out.

Out-of-plane displacement of a wall results with; tilting first developing horizontal cracks, separation of the outer leaves of two or three leaved wall, vertical cracks at wall intersections and overturning of the walls. Same happens on the corbelled vaults and domes, first

develop crack along their base (Figure 4) the Non-load bearing walls, especially gable walls collapse first. Walls not properly connected to the transverse walls separate or collapse at the corners.

In cavity walls if the inner core and outer leaves are not connected with suitable lacings, the wall cannot transmit horizontal forces and outer leaves can separate, rotate and overturn.

The geometry of the structure is important in resistance to the imposed loads. Octagonal or hexagonal layouts are not suitable in seismic areas. No matter in which direction the earthquake forces are imposed, there are walls not orthogonal to the direction. The towers or chimneys within the building complex might experience extensive damage and partial destruction due to the substantial difference in the natural frequencies and high slenderness ratio.

Diaphragm action requirement that couples the out-of-plane and in-plane motions of perpendicular walls, differs in thick walled and thin walled masonry. Limiting the displacements of massive, thick out-of-plane walls may cause excessive damage to the in-plane walls.

The principal purpose of the floor and roof systems in a thick-walled building should be not to re-distribute the loads, but to provide stabilizing forces to the out-of-plane walls. Horizontal floor and roof diaphragms play a critical role in the seismic resistance of thin walled masonry construction. If the diaphragm system is continuous along the walls and well-fixed to the top of the wall, it can limit the movement of the cracked wall and improve the in-plane performance.

Under earthquake shaking; beams may be moved from their position; brick arches become weak with out-of-plane bending and in-plane forces; dynamic interaction

may occur between the brick and steel/timber beams of the roof and floors.

5. Foundation Settlement

Differential settlement of foundation is due to change of the underground water level, deep excavations near the site or demolish of the substructure system. These problems may manifest; crushing of the material, in-plane shear cracks, rigid body motions (rotation), overall instability that led to collapse of the superstructure. Large cracks and rotations due to differential settlement of the foundation may be dangerous.

Differential settlements in walls may generate in-plane shear deformation that result in diagonal cracking. Lintels and spandrels over openings may easily crack. Even at significant settlements, the structure may still be stable, if no out-of-plumbing of walls is produced. If the interlocking of the walls is weak, or if the walls are subjected to out-of-plane thrust due to vaults or roofs, the walls may generate outward rotation. Excessive out-of-plumbing may lead to instability and collapse.

Arches in equilibrium show large capacity to accommodate to even very large differential settlements. Differential foundation settlement may result with the movement of the supports in arches. If the depth of the units constituting the arches are deep in relation to span, this movement results with hinges.

The structure will be safe while the number of hinges keeps lesser than three for the mechanism (Figure 5). Arches, vaults and domes collapse when the supporting walls lose the ability to resist exerted loads. Movement of the supports of corbelled domes and vaults develops cracks parallel to the supports.



Figures 5. Hinges on the arch.

6. Wrong intervention

Wrong interventions may be discarding the local construction methods or using incompatible materials or restricting the deformations in restoration.

Wrong interventions may cause masonry wall to fail during earthquake.

Restriction of deformations to continuous or cyclic forces in masonry construction may develop additional loads.

Applying shotcrete on walls, anchoring the steel ties with cementitious material or changing the flooring to reinforced concrete flooring give additional loads to the structure. Hammering of the reinforced concrete floor during earthquake may cause the collapse of wall under that floor (Figure 6).



7. Final Remarks

In historic structures, it is important to accurately investigate the causes of the damage and know the construction practice of the time.

Identification of the damage and deterioration in these structures is very important for intervention decisions.

For a sound intervention decision, causes contributing to damage must be accurately investigated with the help of historical research, inspection, experiments, monitoring and structural analysis.

Historically stabilized and still active effects have to be distinguished and significance of damage in terms of loss of strength, durability has to be carefully determined [3].



Figure 6. Hammering of RC roof.

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Diagnosing, Pathology, Conservation of Historic Structures

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Abstract

The contribution presents a review of selected methods and approaches applicable for material and structural diagnostics of architectural heritage as well as for their failure analysis and damage mitigation.

Keywords: Historic materials, Historic structures, Failure analysis, Testing, NDT, Damage mitigation

1. Introduction

International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage of ICOMOS created an international group of experts who prepared a series of lectures for the First International Workshop on Restoration and Strengthening of Historical Structures, which was successfully organized by the Iranian Cultural Heritage, Handicrafts and Tourism Organisation in Tehran on 20-22 October 2018. This contribution presents an expanded layout of the author's lecture focused on a brief review of modern methods for structural and material investigations, damage analysis and damage mitigation or prevention. The text is divided into blocks grouping the main lecture themes.

Structural health and condition assessment of historic structures is characterized by several typical needs:

Safety evaluation for retrofitting or strengthening measures; Investigation of defects and failures; Checking degradation or impact of an intervention and Redesign for changing loading or functions.

The needs listed above call for typical actions: Testing of material characteristics and performance under loads; Testing of structural characteristics and performance under load; Long term monitoring; Investigating long term loading or environmental effects.

2. Testing of historic materials

2.1. Testing of material characteristics

Material testing provides data for numerical analysis as well as for planning and evaluation of structural testing (e.g. proof load tests). At planning of tests we need to consider what type of tests will be adopted – namely in-situ tests on real structures or laboratory tests on test specimens manufactured from material samples taken from historic objects. Further, we should know how the material testing data will be interpreted and transformed into design material characteristics. The

planning of tests is also influenced by test conditions, e.g. by an allowable number of test specimens or by environmental situation. Testing needs tools for checking the representativeness of the identified material characteristics – e.g. influence of imperfections – by means of comparison of deformations determined by numerical modelling and measured during structural tests. The task is frequently determined by specific features, for example by a requirement of testing material characteristics after consolidation or strengthening interventions.

2.1.1. Number of material samples

This is a very important issue for historic structures. Many times standards for material testing are used also for historic materials as supporting tools and they typically require large amounts of testing specimens, which is not possible to accept according to other – conservation standards.

In Europe the Standard EN 16085 (August 2012) “Conservation of Cultural property - Methodology for sampling from materials of cultural property - General rules” defines basic approach to extraction of material from historic structures, buildings or other objects. No specific rules or numbers of samples are suggested there and all interventions must be individually designed by the assessor in a plan of sampling taking into account the condition of the structure and the objectives of sampling. However, minimum intervention is a general rule and non-standard testing is usually applied as the only possibility.

2.1.2 Why non-standard testing

Non-standard testing enables direct determination of

physical (namely mechanical) characteristics by means of minor or considerably destructive tests (from the object conservation point of view) on test specimens of non-standard dimensions. This type of tests includes also laboratory testing of materials extracted from a historic structure or purposely designed for testing of new agents.

One of the important reason for such an approach lies in the fact that engineers prefer destructive tests on a real material in contrast to the in situ mostly non-direct testing because it provides data for a better possibility of numerical modelling and gives a possibility to follow up non-homogeneous change in material qualities.

2.2. Mechanical characteristics of mortars and stone

The tests are mostly done in laboratory on specimens extracted from structures. The specimens are typically i) cut from irregular bodies, ii) cut from bedding joints, iii) split from core samples from masonries or quarries for stone, iv) detached from floors - grinded and levelled or v) cast for special study reasons.

The volume of sampled materials is usually not enough for manufacturing standard testing specimens and we need to use smaller size specimens. Specific techniques are available for such material testing [1], [2].

2.2.1. Compressive strength testing

Production of standard cubes of the dimension of 40 x 40 x 40 mm³ is rarely possible. Therefore, flat plate like specimens are frequently used for the tests. Experimentally measured strengths on flat specimens are however significantly higher than values measured on

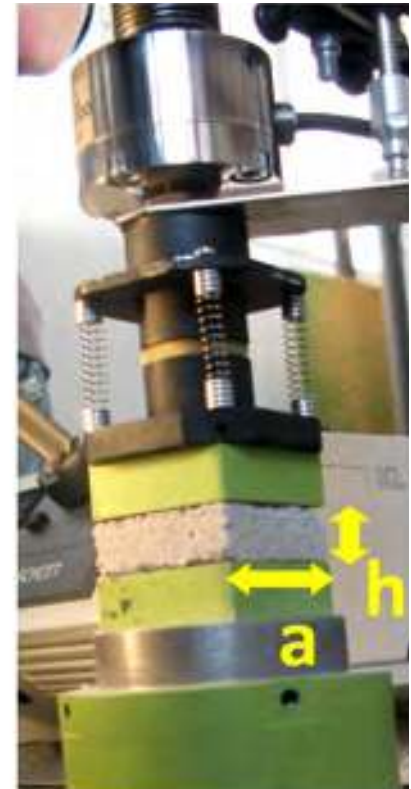
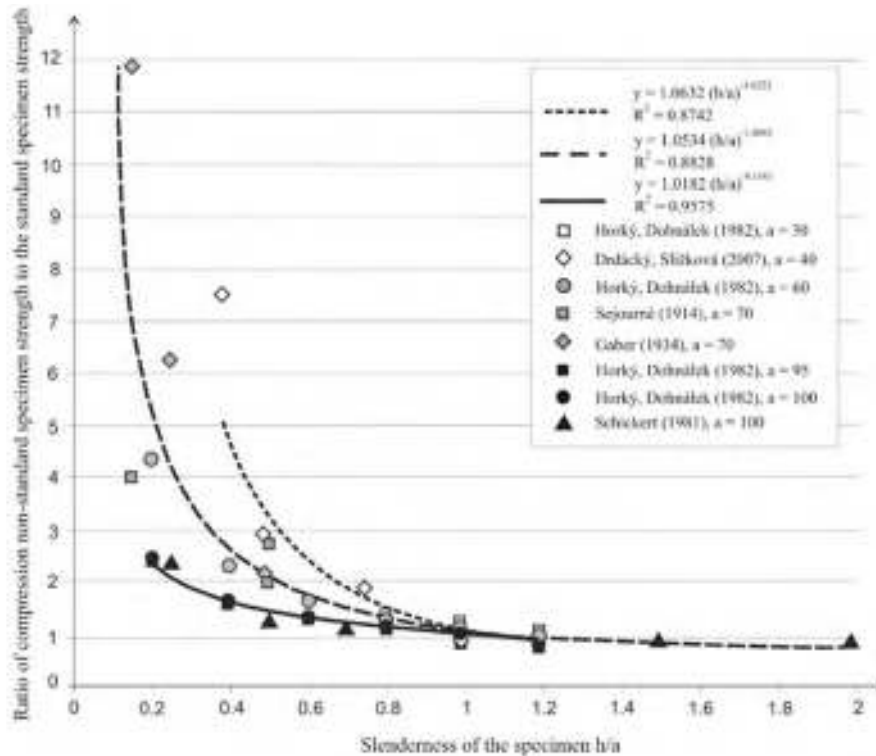


Figure 1. Compression tests on flat specimens from cement based mortar.

standard cube specimens and they are influenced by: i) slenderness of the specimen (height h (thickness) / min.length a of the base edge ratio), ii) friction between the specimen and the grip plates, iii) length of the loading plate or the base edge, iv) mortar quality and v) type of testing (double punching / equal plates). The measured data – compression strength - can be translated into standard equivalent cube test data using correction formulas in literature available, (see Kashkan Bridge report), or experimentally achieved for different types of materials, e.g. for lime mortar in [3]. Examples are given in Figure 1.

2.2.2. Flexural strength test

The shortage in the bulk of material necessary for standard bending tests can be overcome using a special technique of production of beam like specimens from smaller prisms prolonged on both ends with prostheses made of other material, usually from wood, Figure 2. Wood is light, well workable and bondable and, therefore suitable for such a technique. Naturally, there is also a possibility to use beams from mortar or stone of non-standard dimensions, e.g. with cross section only $20 \times 20 \text{ mm}^2$. In such a case the size and shape effects must be considered and the measured strength

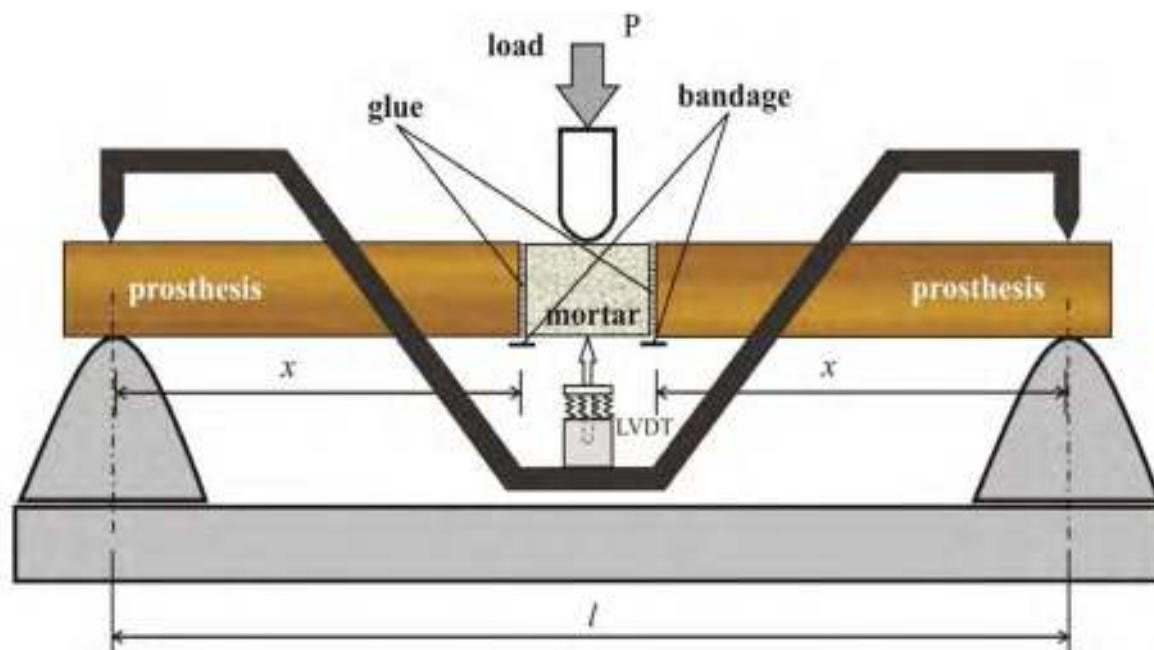


Figure 2. Flexural tests on flat on prolonged specimens.

corrected with experimentally attained formulas. The smaller cross-section the higher strength is measured, and the value is further influenced by the maximum size of sand grains in the mortar, [2]. From the flexural tests the Young modulus can be calculated if the deflection is recorded.

2.3. Mechanical characteristics of built in timber

The mechanical characteristics of wood built in timber structures can be also tested in laboratory on small sample – non-standard testing specimens. However, for wood testing there are available several other methods for non- or considerably destructive tests on real structures in situ. The translation of the measured data into engineering data needed for

redesign purpose is explained for example in the paper [4].

2.3.1. Compression along fibres

For laboratory tests of compressive strength of wood when loading along fibres a so call “core drilling” method has been developed. It consists in an extraction of small diameter (around 4,5 mm) cores using a hollow drill. Then the cores are tested in a special rig in which they must be placed with fibers perfectly aligned with the direction of the loading force, Figure 3. The correlation between the strength of radial cores and the strength of standard specimens determined in compliance with EN 408 range in the interval $R2 = 0.7–0.9$ based on the wood species. Modulus of elasticity can be directly assessed from the elastic part of a stress – strain

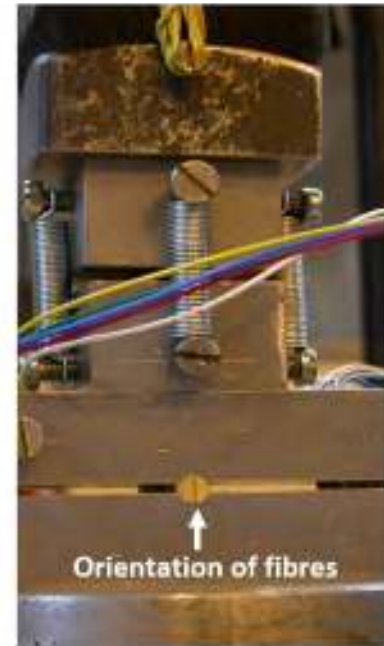
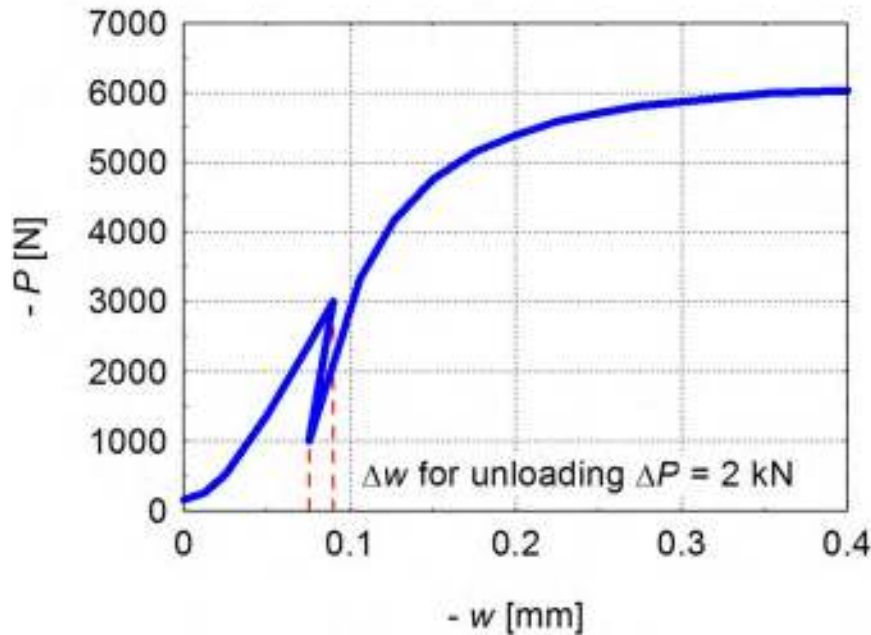


Figure 3. Compression tests on small wooden cores.

diagram measured after unloading the specimen during test, which is schematically presented in the Figure 3.

2.3.2. Tension tests on historic timber

For tension tests thin strands of wood are extracted from the surface of timber elements using kerf saw guided in a special rail and cutting the specimen in two inclinations along the grains (fibres) direction. The resulting strand of triangular cross-section has a typical dimension of sides about 7 mm [4]. After ending of the strand with wooden blocks the specimens are ready for tension tests which are very close to the standard tests, Figure 4.

2.3.3. In situ testing of wood characteristics

As an example of the most modern device for in situ

testing of wood mechanical characteristics we present here a mini jack which enables compression loading of wood in a 12 mm diameter hole. The loading is carried out in natural environment of the structure, the device allows for measurements in various depths along the timber profile and provides the engineer with classical stress/strain diagram, Figure 5. [5], [6]

2.4. Special testing for conservation interventions

The conservation field requires specific testing procedures for purposes beyond the structural needs. As an example, let us present a set of typical tests for stone conservation. Again, the amount of historic material which is available for the tests is limited and the restorers need to know various material characteristics

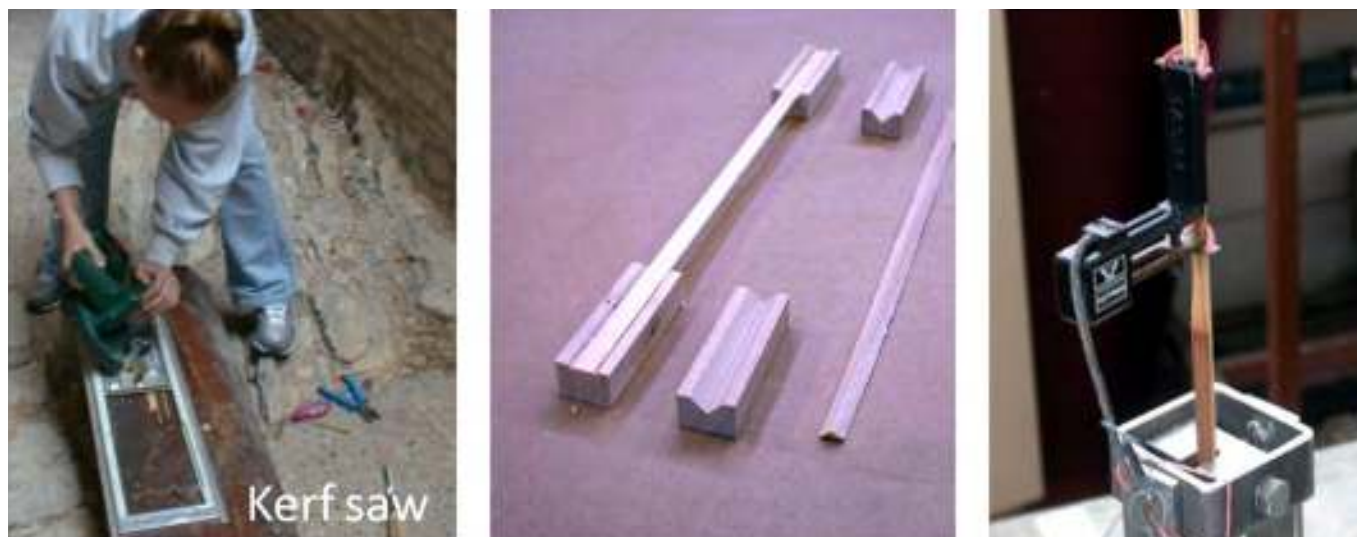


Figure 4. Extraction of strands for tension tests and mounting of a strand in a hinged rig.

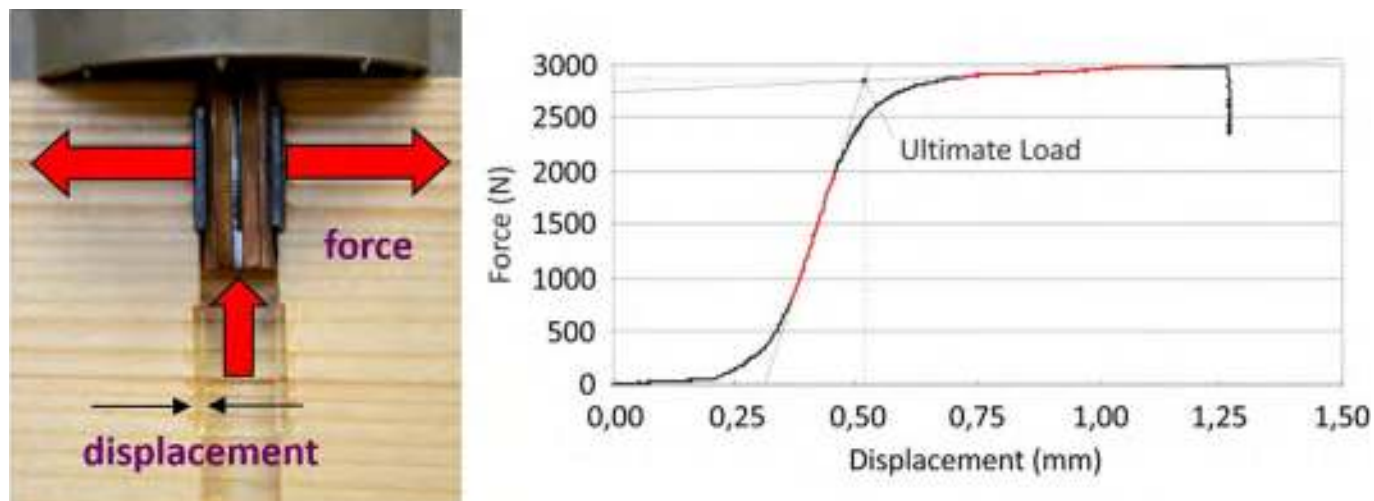


Figure 5. In situ measurements of wood strength in compression along fibres.

for an appropriate selection of the restoration process and materials. Laboratory tests are usually completed with in situ tests before and especially after the restoration in order to check the efficiency of the intervention.

2.4.1. Stone testing for conservation planning

A set of tests on thin stone plates for conservation purposes is presented in Figure 6. The sequence of tests is optimized and enables to perform all tests on

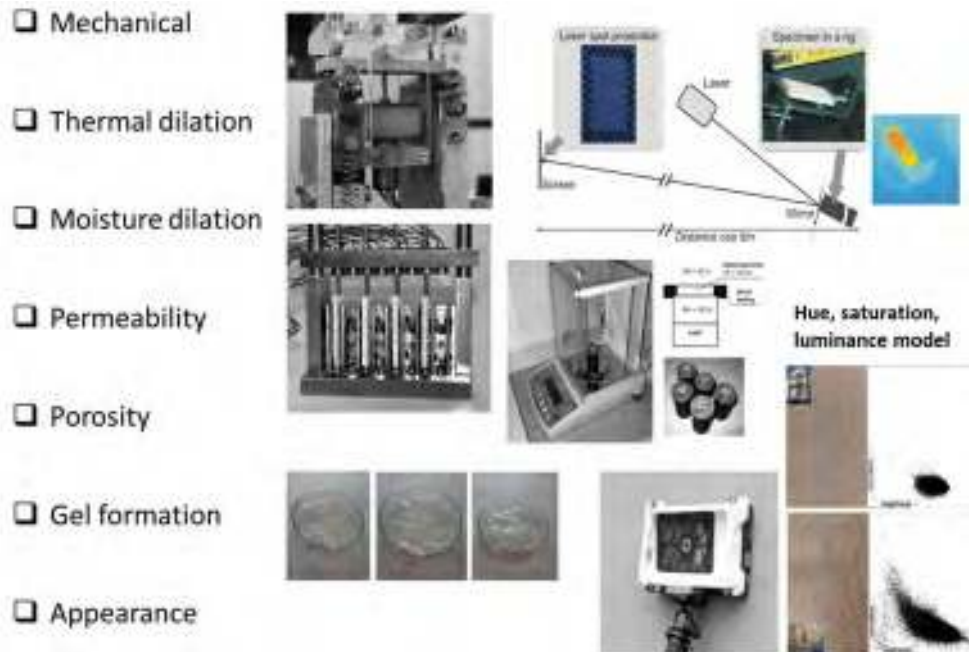


Figure 6. Set of typical tests for stone conservation planning.

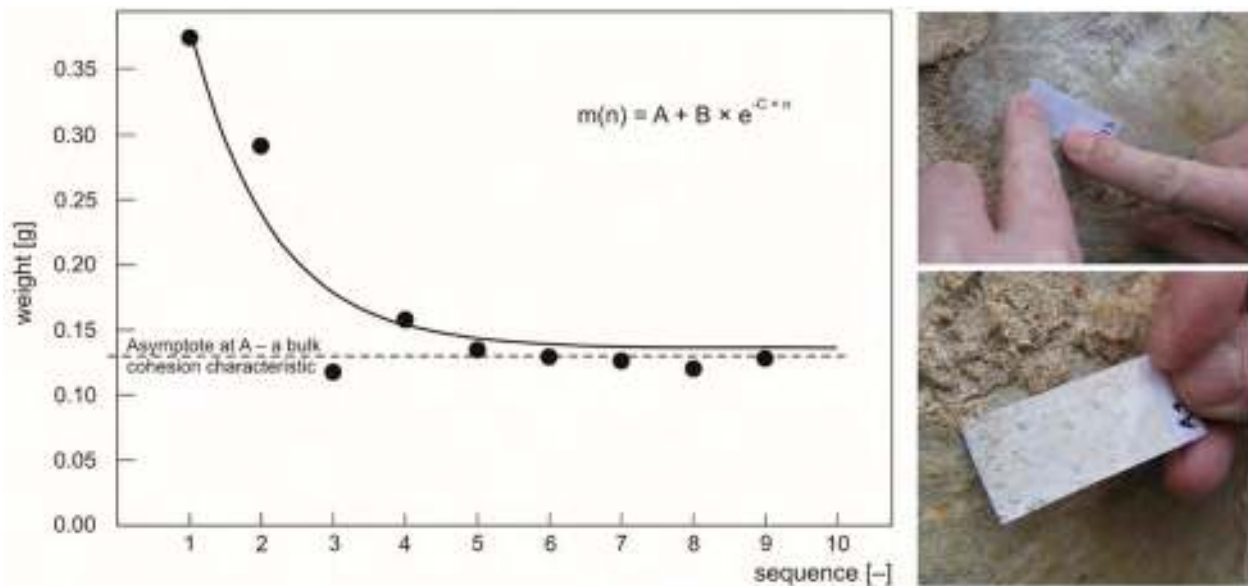


Figure 7. Peeling test for assessment of surface cohesion and consolidation effects.

identical specimens, which decreases errors due to material non-homogeneities. It includes also tests of consolidation agents.

2.4.2. Stone and mortar surface testing after conservation intervention

Consolidation effects are tested in situ on the surface of plaster or stone. A rather simple and affordable method takes advantage of so called peeling test. It consists in attachment of a short tape with sticky layer – typically so called “Scotch tape” – and its removal after about a minute. Surface particles attached to the glue layer signal the degree of surface cohesion and can be used for assessment of the consolidation efficiency. During the testing, it is important to repeat the peeling several times on the same place in order to eliminate the influence of dispersed individual surface particles which are not representative for the cohesion.

The typical result of such tests is shown in Figure 7. [7]

2.4.3. Testing of penetration depth

For in-situ measurements of penetration depth and consolidation effects by GEOTRON developed in cooperation with ITAM an integrated two probes device. An US transmitter and a receiver are to be inserted into holes of 20 mm in diameter drilled into the surface layer under investigation at a distance up to 100 mm and a depth of up to 60 mm. The device is portable and fully compatible with laboratory US equipment. [8]

3.3. Pathology of historic structures

3.1. Surface pathology on heritage objects

Critical and decisive parameters of surface layers are

physical – loss of cohesion, local failures, water and/or vapour permeability – porosity, cracks; chemical – salt crystallization; biological – bacterial soiling, algae etc. and complex effects – appearance & aesthetic perception (important for calls for renovation). In time repair requires regular inspection & monitoring and assessment of façades for decisions.

For the stone deterioration patterns an illustrated glossary issued by ICOMOS International Scientific Committee for Stone Conservation in several language variations is available. In Farsi it was translated by Dr. Manijeh Hadian, Parisa Abdollahi and Rasool Vatandoust. Surface layers made with mortars have specific features and require a sufficiently descriptive terminology and an analysis of their defects. Under the term “surface layer” we understand a layer or a composed system of layers firmly attached to a substrate that usually possesses different material qualities and characteristics. It follows from the nature of monuments that the composed layers may include parts of the original material that have been transformed on one surface by physical or chemical changes.

Since surface layer defects and failures are tightly connected to their structural arrangement, the classification system needs to distinguish between failures of i) thin surface layers, e.g. paints or paint systems (thickness < 2 mm), ii) single-layer plasters, (thickness about 10 mm), iii) multilayer plasters, (thickness about 25 mm – may reach 100 mm), iv) lining layers, (thickness < 40 mm), v) mosaics, (thickness < 40 mm) and vi) plasters (gypsum) with plastic decorative elements (about 20 mm).

The defects and failures can be classified into six basic groups, similarly to the defects in stone surfaces, as

defined in the ICOMOS Glossary for Stone Degradation. However, combinations of various types of defects are very common for mortar surface layers, for example detachment and loss of cohesion of subsurface layers, cracks and detachment, which may then generate a series of variants in the defect morphology. Distinctions can be made between defects characterized by loss of cohesion or blistering in closed or open forms, smooth shapes and folded shapes.

For the purposes of this paper a further step that involves analyzing or even revealing possible defect initiation is considered more important than a very detailed complex phenomenological description. Some examples [9]:

3.1.1. Cracks

For example Dilation cracks – these are typical for surface layers made of materials in which temperature and/or moisture dilation generate significant volumetric changes or different behaviour in relation to the substrate, frequently combined with structural cracks. In contrast to technological cracks, dilation cracks are not stable. The cracks are typically old and usually dirty, which is reflected in their appearance. Because they are a natural system response, it is recommended not to “stitch” them firmly or fill them with stiff material.

3.1.2. Deformations

Buckling - deformation of a larger integral plate of a surface layer in the shape of a smooth or folded (saddle roof) buckle or bulge – close or open. Buckling is always combined with detachment, due to the compression forces within the layer.

Bowing - deformation of a comparatively large integral

plate of a surface layer in the shape of smooth buckle or bulge, typically combined with detachment, bent due to loss of stiffness or loss of strength. A high level of tension stresses is present.

3.1.3. Detachment of layers from the substrate (typically combined with deformation)

Blister – this denotes a raised portion of a surface layer in a smooth or folded (saddle roof) form - which may be closed or may have open peaks. Blistering is accompanied by a local loss of cohesion with the substrate or loss of integrity of the layer under the blister.

3.1.4. Buckling generation

There are several mechanisms for buckling generation: i) initial detachment (salt crystallization, vapour pressure, local defect of cohesion with substrate, repeated shear stress, ...) or ii) differential mechanical response to vertical loads – the surface layer is stiffer than the substrate – various reasons, e.g. PC mortar in surface layer unlike lime or clay mortar in masonry, or creep in masonry, different aggregate, ..., or iii) differential and gradual expansion of the surface layer – e.g. due to salt deposits in the surface layer or its part, gradual loosening of packing of the composite (grains) in the surface layer, which may be caused by repeated salt crystallization in the surface layer, frost damaging, repeated thermal or moisture dilation.

3.1.5. Blossoming

There is a local deformation of the separated surface layer of the cracks in a form that resembles flower leaves, with wound leaves, accompanied by a loss of cohesion with the substrate along the edge, and later

loss of cohesion of the whole area of the detached leaf, Figure 8. Conservation intervention depends on the cause of the damage - in thin layers, the damage is typically due to a residual stress in the surface layer, while in thin layers of mortar blossoming is usually due to expansion of the detached interface layer due to gradual disintegration.

Blossoming may generate due to: i) initial cracking (residual stress due to polymerization, hydration or carbonation shrinkage) or ii) initial detachment (vapour pressure, local defect of cohesion with substrate, shear stress on the interface, ...) or iii) differential and gradual shrinkage of the exterior part of the surface layer – e.g. due to action of accumulated residual stress on released parts or change of polymer structure in the surface layer, or iv) differential and gradual expansion of the interior part of the surface layer – e.g. due to salt deposits in the surface layer or its part, gradual loosening of packing of the composite (grains) in the surface layer, which may be caused by repeated salt crystallization in the surface layer, frost damaging, repeated thermal or moisture dilation.

3.1.6. Detachment

Detachment (loss of adhesion of a layer to the substrate) - this is an interruption of continuity and integrity near the interface between the substrate and the surface layer.

The surface layer responds to knocking with a typical “dull” sound. It can take the form of bursting an integral thin floe, namely when very hard and strong paints or plasters are used. Usually caused by differential dilation effects.



Figure 8. Examples of surface layer blossoming damage.

3.2. Structural diagnostics

3.2.1. Detection of subsurface detachment areas or structural defects

Active thermography is based on the idea of measuring differential cooling or heating of loosened parts compared to the heavy and integral parts with higher thermal inertia and different conductivity. In the environmental excitation variant, differences between daytime temperatures and evening or night temperatures are exploited. After a warm day, the outdoor temperature drops in the evening to values which are sufficient for natural cooling of the investigated surface. Passive variants is useful for identification of a structure of plastered masonry, structural details or metal elements in masonry, for example Figure 9.

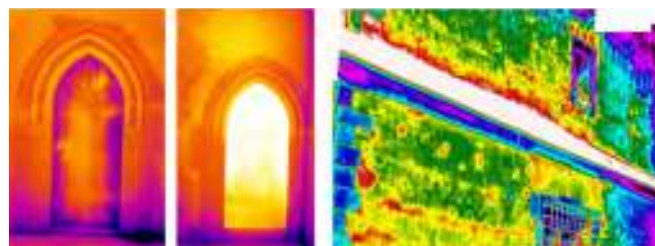


Figure 9. Examples of visualization of hidden masonry structures.

3.2.2. Ground penetrating radar (GPR)

This non destructive geophysical method exploits pulses of electromagnetic radiation in the micro-wave range. Short pulses of electromagnetic waves are transmitted into ground or masonry. Signal reflected and changed on interfaces of individual layers of materials with different dielectric constant is registered by the receiver antenna.

Figure 10. shows radargram of a masonry vault. [10]

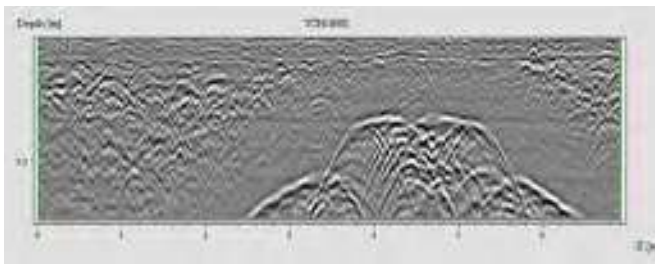


Figure 10. Cross-section of a masonry vault.

3.2.3. Monitoring

Monitoring of historic structures is a powerful tool still underestimated and rarely used even though very effective mainly in the following applications: i) to follow up behaviour of existing damages and failures on buildings, ii) to predict and to record early failures in structures, iii) to provide data for estimation of safety and remaining life of a building or a structure, iv) to signalize defects and needs for maintenance or repair interventions, v) to provide information concerning designed or expected and real structural behaviour. Accordingly, the monitored parameters (state indicators) are to be selected with respect to their meaningfulness in describing the observed phenomenon. Historical masonry structures are often complex systems consisting in several heterogeneous components.

A monitoring of all components with sensors is economically and technologically not feasible. For this reason, an effective monitoring procedure should be subjected to a cost-benefit analysis so that only data strictly necessary to reveal progressive phenomena are collected.

Structural pathology is frequently dependent on environmental fatigue phenomena, therefore, monitoring of cracks should be always accompanied with monitoring of temperature and moisture fluctuations. There are recommended continuous (electronic measurement based) monitoring systems in front of a simple indication of instability, e.g. gypsum or glass plates.

4. Damage prevention and resilience

4.1. Material consolidation

Consolidation of historic degraded stone and mortars requires tailored solutions. It should be based on use of compatible materials and technologies. Next to the traditional materials (lime or gypsum binder) also modern materials and technologies are acceptable. [11] Innovative compatible materials - nanolime based sols consolidation agents represent a powerful tool in this case, Figure 11. The advantage is seen in adjustable concentration of the $\text{Ca}(\text{OH})_2$ particles in alcohol (5 g/l – 80 g/l), usually applied in 2-3 cycles. It exhibits a very good wettability, low surface tension helps to penetrate in stones and mortars (namely into gross pores and cracks), it has a fast carbonation (ideally at $\text{RH} > 75\%$). There are problems with long term stability of sols (coagulation) & a rather high price. In stone conservation nano-materials are used advantageously for consolidating disintegrated historic

materials, healing cracks and fissures, materials with complex problems, namely wet and chemically contaminated materials, or materials in complex situations requiring complex solutions, e.g. combined consolidation and disinfection. Further research includes stable sols based on $\text{Ca}(\text{OH})_2$ - $\text{CaSO}_4 \cdot 1/2\text{H}_2\text{O}$ mixtures for consolidation of stucco and consolidation of gypsum or gypsum mortars. Promising are also combinations of nanosols and ethylsilicates.

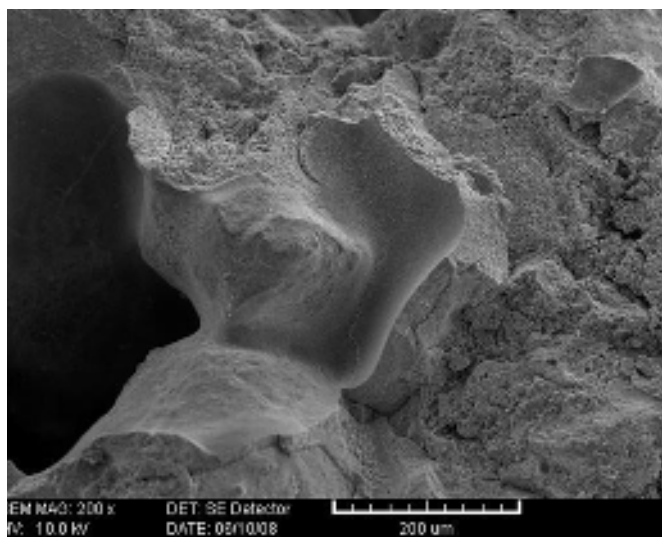


Figure 11. Nano lime bridging of mortar grains.

4.2. Mitigation of damage from natural disasters

Major natural disasters problems in Iran are connected with flood, earthquake and weather effects. Historic structures under flood situations are subject of various loads and actions which can be categorized as well as the respected vulnerability of architectural heritage [12]. It helps to adopt measures mitigating flood impact on sustainability of heritage objects. [13]

4.2.1. Measures against horizontal static pressure of high water

Territorial protection is the most effective using prepared technical means against floods – permanent or temporary ready to be easy installed. Temporary strengthening or additional supports help to free standing walls, sculptures as well as doors and windows (Figure 12). Measures reducing loads is another effective approach, namely temporary removal of objects



Figure 12. Example of temporary protecting wall.

out of the endangered areas or positions, opening (creation of) inlets for balancing water pressure or optionally filling of the underground spaces with water.

4.2.2. Measures against vertical hydrostatic pressure

Technical means include shutters against high water in sewage systems, shutters of channels and underground corridors of all kinds (protection against water penetration). Further, enhancement of anchoring of sensitive

structural elements in supporting structures and objects prone to uplifting (light bridges, roofs).

The measures include also prevention of creation of barriers blocking accessibility after the flood, for example unhinging of doors (or fixing against unhinging) and stabilization of object against overturning (usually furniture).

4.2.3. Measures against dynamic action of slow water stream

Protection of surfaces against washing out is one of the most urgent action, e.g. by wrapping” of masonry with clay mortars with a sheet of plastic foil. Anchoring of objects is another important measure for protection of easily floated objects (wooden, plastic foam, ...). It is completed with stabilization of objects prone to overturning – it concerns also retaining walls where the buoyancy decreases gravitation stabilization efficiency.

4.2.4. Measures against barriers and impact of floating objects

Removal of floating objects from the water stream (catching and bring to light), checking of anchoring of boats and floatable objects, (e.g. cottages) and a regular maintenance, removal of obstacles in a water stream, (e.g. dismantling of bridge rails or rail walls) and blasting of barriers.

4.2.5. Measures against destabilization of subsoil and foundations

This measure is historically effective in a form of paved islands or continuous pavements around the foundations of pillars (Figure 13).

4.2.6. Measures against failure of wet masonry

Dried brick masonry rendered with water resistant plaster (even only lime mortar) survives flood without problems, dried brick masonry must be protected

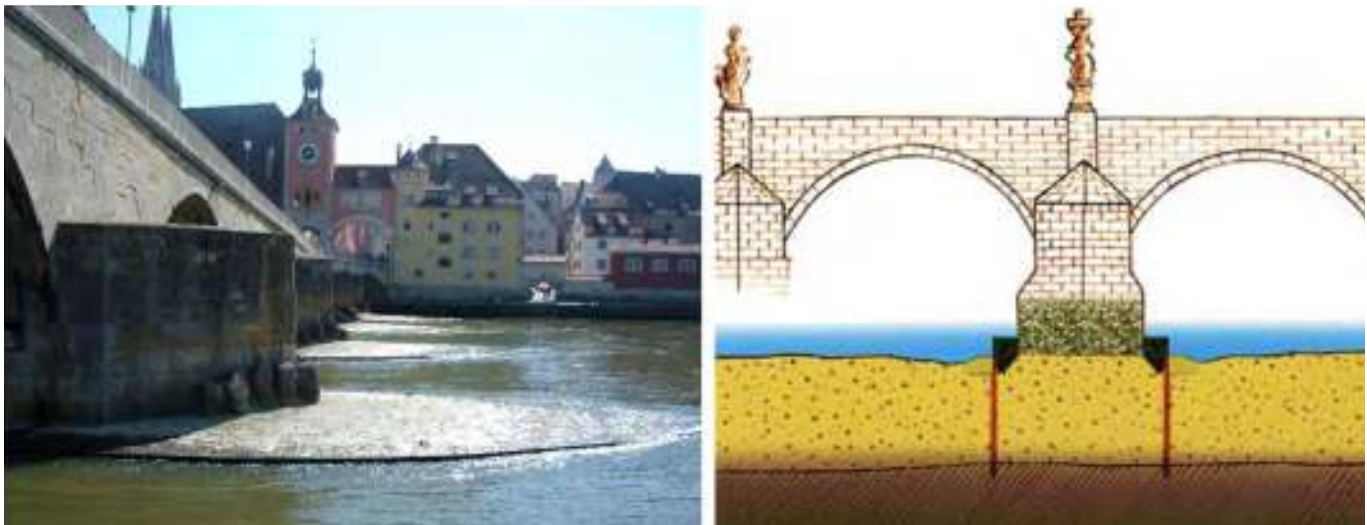


Figure 13. Examples of historic and modern protection measures of bridge piers against undermining.

against washing out by water stream (even slow), it is recommended to protect adobe without resistant plaster against direct contact with water; chemical protection is also possible, e.g. surface impregnation with gelatine or ethylsilicates.

At masonry with clay mortar, it is recommended to apply the same as for the adobe.

Burnt brick masonry (and water sensitive stone masonry) need temporary supporting of weak points (e.g. pillars) because these materials are very sensitive to wetting. Moisture may decrease the strength up to 50%.

4.2.7. Measures against defects due to timber swelling and degradation

Creation of dilation (expansion) joints prevents damage on adjacent masonry - wood anisotropy must be considered – dimension changes along different axes – longitudinal (ø 0,15%), radial and tangential are different. Temporary support of degraded timber structures is recommended. All measures should be based on regular inspection and maintenance.

4.3. Importance of regular maintenance

Buildings are particularly vulnerable to damage when no/poor maintenance has been carried out for years. This frequently happens in the case of vernacular or minor historic buildings. Some examples: It contributed to the failure of poor houses in Haiti or villages around L'Aquila, but also of San Francesco in Assisi, flood in Prague Troja, and in windstorms, too.

Regular maintenance is one of the most important strategies against damage. Lack of regular maintenance leads to material decay and loss of the structural properties. Maintenance is usually a result of a regular in-

spection, or regularly on a basis of a maintenance plan [14]. Maintenance actions in most cases do not require design work or even engineering supervision. They can be left to the skills of properly trained craftsmen, which enables action to be taken quickly, and prevents a defect developing into more serious damage. A maintenance guide is a useful tool, and should combine tips for inspection with recommendations on how to fix problems that are identified [15]. In the seismic protection the work should focus on the state of the joints, masonry integrity, material condition, resistance, overall integrity & stability.

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Intervention Methods

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Abstract

The paper is enthusiastically supporting the initiative for conservation and rehabilitation of the Iranian Cultural Heritage. For choosing the most appropriate methods of intervention the typology of monuments, as structure or massive, and their gravitational nature, dead or live, should be first, precisely known. Every intervention should start from the cause of damage or failure, and not from its effect. The foundation of the building should always be the first considered for examination with maximum priority. A detailed hydro-geological report cannot be omitted. The estimated durability of the proposed intervention might be a good criterion for quality control. For responsibility reasons the decision of intervention should be lucidly taken by one's individual mind, not collectively, and always also checked by one's own heart.

Keywords: Adaptation, Confining, Dislocation, Durability, Flexibility, Reliability, Rigidity, Safety

1. Introduction

1.1. Ancient civilization

The Persian territory seems to be the place where

human civilization was created. All cultures that have ever existed on the Earth were born from the original Persian civilization. From Persia the first human cultures were spread all over the world. All arts and human professions, mathematics, music, astronomy and astrology, cadastral survey, medicine and agriculture rose from this place. There are many vestiges and proofs that confirm the former civilization: a large network of roads and viaducts with bridges, monumental buildings and memorials, astrological centers, castles and palaces, to remain in the field of civil engineering only. They are however too few to disclose the proportions of the ancient Persian civilization. This is why the existing vestiges should be carefully preserved and, where necessary, accordingly restored.

1.2. Tehran Workshop

Through ISCARSAH the Workshop was put under the authority of UNESCO, as a Governmental Organization, and also of the World Heritage as the base upon which humanity constructs its memory and its identity [1]. ISCARSAH is an International Scientific Committee affiliated to ICOMOS, the International Council on Monuments and Sites. ISCARSAH Recommendations

are devoted to the analysis, conservation and structural restoration of architectural heritage: Part I Principles, Part II Guidelines and Part III Glossary. ISO 13822:2010, Bases for design of structures – Assessment of existing structures, is strongly supporting the ISCARSAH Recommendations [2].

1.3. Intervention methods and the purpose of this paper

Before the discussion about intervention methods, the ISCARSAH Recommendations ask that four distinct steps should be followed, surely regarded in the precise conditions provided by ISO 13822:2010: 1) Anamnesis, 2) Control, 3) Diagnosis and 4) Therapy. The same ISO explains how finally, with the aid of the plausibility principle, the most appropriate intervention methods are carefully analyzed and then, by using the principle “the one who pays is the one who decides”, one method is chosen, and the legal procedure of selection ends. For this paper the author’s intention is to emphasize the important role of the anamnesis as it was provided by the Recommendations.

Then the terminology used in all official documents to describe the ancient buildings and works, should be carefully chosen, in order to be accurate. The meaning of the terms used needs to be precise without creating confusions, doubts or misunderstandings. Since long ago the Greek philosopher Plato (c.427-347 BC) was greatly concerned about the dramatic distortion of reality that is unconsciously produced by the human mind [3].

For Buildings and Works of Cultural Heritage such mistakes may have irreparable consequences and therefore cannot be accepted. This is why the paper further pres-

ents some case studies which might be useful in understanding the philosophy of the intervention methods.

2. Lesson of *Æsopus* (c. 620-560 BC)

The fable of the reed and the oak is well known worldwide. Both enjoyed standing relaxed during good weather. Once, it happened that a strong storm occurred. The wind started to blow furiously. Carefully, under the wind pressure, the flexible reed immediately bent as much as was necessary while disdainfully the stiff oak remained standing, opposing to the storm. Shortly it was uprooted and thrown down along the soil. After the storm passed the damaged oak saw the reed standing in good shape and asked how it escaped? Simply, came the answer, by bending when it needed. The moral of the fable is that flexibility is preferable to rigidity or stiffness. The role of the underground should also be considered. In Civil Engineering the elastic structures, which are bodies with 1D or 2D, are symbolically represented in the fable by the reed while the inertial, massive bodies with 3D, are symbolically represented by the oak. Regarded from an historical perspective, the existence of the two pairs of bodies and symbols raises a natural question: when were they created, simultaneously or successively? Since obviously the first alternative is unlikely, the resulting dilemma is which one was the first, the reed or the oak, and why, for which reason? The answer to this hot question is given by the legend that follows.

3. Legend of *Oedipus*

At some point in history, not at all by chance, but very accurately in its evolution process, *homo erectus* was endowed with consciousness by the cosmic mind [4]. That

means the activation of the instinct by which the human became aware by its own existence, but unfortunately without knowing its own cosmic identity or at least getting a slightest idea about it. Obviously, it was a special event, it has occurred unexpectedly only once and out of any control in human history. According to the legend, it was necessary that a winged lioness like a monster should check each individual, under the threat of death, whether or not consciousness was successfully induced. Oedipus was the Greek Prince who answered correctly to the monster's riddle that humans are controlled by the force of gravity permanently, day and night and from birth to death. It is an intrinsic propriety of the matter and it is governed by the universal law of mutual attraction established by Isaac Newton in London, in 1686 [5]. Thus, Oedipus became the first *homo gravitas* in the world, and the answer to the above mentioned dilemma is the inertia force because inertia is a latent propriety of the matter [6]. From the very beginning, this new *homo gravitas* started an extensive program for searching about its own identity with all the means available then. As expected it did not find anything relevant. But after a while *homo gravitas* erected the Sphinx in Egypt, a monument supposed to be devoted to the Four Elements for the first time, locked into place, and matter finally became solid. During that time, in the evolution process, individual consciousness was enriched with two additional components called sub- and supra-consciousness.

4. Pyramids of Egypt

In the obsessive search for its own identity, the primitive human, already having a status of *homo gravitas*, stylized its body as sitting in a meditative position, in the

gravitational field; with a squared pyramid. Then the pyramid was built up as a funeral monument, in stone or usually sandstone, which was easily available and manually detachable from the site. The first such pyramids were erected on the Nile Valley in ancient Egypt before the middle of the third millennium. They consisted of brute pieces of stones, manually laid one over the other without any binding material between them. The stability of the nonhomogeneous construction such created was ensured by the gravitational forces of the stone weight and the dry frictional forces. The final shape was that of a regular pentahedron with four vertical plans of gravitational symmetry. This configuration satisfies Euler's topological theorem between faces F , peaks P and edges E , namely $F+P=E+2$. The pyramids were idiomorphically shaped as solid-rigid bodies with equal strength to vertical centric compression from the gravitational action. It is worth to notice that these stone pyramids are nonhomogeneous massive bodies, but not at all monolithic. The pieces of stones are not in full contact between them so that when some slight settlements occur randomly they are free to happen, without being prevented in any way. Local concentrations of stresses are prevented in this way. The maintenance of such buildings is extremely easy and costless. This is why they have lasted almost unchanged. 70 such pyramids have been identified in Egypt so far. The largest one, the so called the Great Pyramid of Giza, was built up by Cheops, probably between 2601 and 2515 BC. Its base is $B=232.8\text{m}$ and height $H=148.2\text{m}$. By building these stone pyramids in common, thousands and thousands of people together, human consciousness was enhanced by three levels, from individual consciousness to individual and collective consciousness,

both with under- and super-consciousness. In author's opinion, this Pyramid of Cheops is the first Memorial devoted by *homo gravitas* to gravity, as the force of life on the Earth.

5. Ziggurats of Mesopotamia

Ziggurats are monumental buildings erected in the Plain of Shinar, between the Rivers Tiger and Euphrates, several centuries after the Egyptian Pyramids. Their functions are scientific and religious, therefore their configurations are no longer pyramidal, but more geometrically elaborated (Figure 1).



Figure 1. Susa Ziggurat (Choqa Zanbil), Iran.

33 ziggurats were identified in Mesopotamia, but this construction system based on masonry was also adopted in other places. The masonry for ziggurats consists either of adobe or solid bricks of clay baked at low temperatures and bound with lime mortars. This masonry is miraculous because it was provided with small vertical joints between the bricks, allowing slight

and spontaneous relative movements between them. In modern terminology, this physical phenomenon is called *adaptation* and it avoids concentrations of stresses around geometrical imperfections. This is an ingenious method to protect masonry against the unexpected consequences generated by the settlements of foundation soil. The ziggurats made with masonry are massive builds, but they are not at all monolithic. Masonry behaves like having local some degrees of freedom and thus is protected against any damages. It is also the case of ancient masonry constructions built in other countries like Turkey, Greece, France, Spain, Portugal, Great Britain, as well as Romania. After thousands of years they are perfectly preserved. The outstanding ziggurat ever built was the Babel Tower or Etemenanki of Marduk Temple, located on left bank of the Euphrates River. Shaped like a regular hexahedron, with seven steps and four vertical plans of gravitational symmetry, the Tower, with a squared base with $B=90\text{m}$ and an identical height $H=90\text{m}$, was built during the Mathematician and King Hammurabi the Great, between 1792 and 1750 BC. After severe damages the Tower was rebuilt by Nabucodonosor the First, between 1125 and 1103 BC. After other damages the Tower was rebuilt by Nabucodonosor the Second, between 605 and 562 BC. In that period Babylon was raised to the rank of Metropole and the Hanging Gardens were built up. The Babel Tower was then considered as a small-scale replica of the Universe. In the year 331 BC Alexander the Great (356-323) was fascinated by the beauty of the Tower, even if at that time its physical state was purer than ever. Seven years later, in 324 BC, Alexander ordered the demolition of the Tower and the construction of a new one. According

to the legend, a mosquito opposed to Emperor's orders in Babylon and the fate of both instantly came to an end. In the year 1898 AD, after 2221 years since the fall of Babylon, the German architect Robert Koldewey (1855-1925) discovered the Tower's foundation. Paradoxically, into a weak and oozy soil on the river's bank it was discovered that the Tower's foundation consisted of adobe masonry, while bricks of backed clay were used for its superstructure. That grave mistake of founding was repeated three times during the Tower's long history. It seems that many other ziggurats have been affected by founding errors. The whole region has important hydrogeological problems that should be further considered by builders very seriously (Fig. 1). The extensive astrological research carried out into the ziggurats of Mesopotamia has brought an immense contribution to human civilization. Human consciousness has been extended then from the individual and collective to the astral one and, together with the two forms under- and super-consciousness, it has reached nine dimensions. Individual identity further remains the great unknown and fortunately, it positively supports the subsequent searches of *homo gravitas*.

6. Persian Royal Road

A network of roads connected by massive masonry bridges was established during the Achaemenid Persian Empire between the years 500 and 130 BC. The Persian Royal Road ran from Susa in North Persia to Mediterranean Sea in Asia Minor. Herodotus (485-425 BC) has used these roads and appreciated the efficiency of Persian messengers. The Achaemenid Empire fell when it was conquered by Alexander the Great in

330 BC and then the Persian Royal Road was closed. Not much later, in 130 BC, the Silk Road was formally established during the Han Dynasty of China and lasted until 1453 CE when it was closed by the Ottoman Empire. Marco Polo (1254-1324) traveled on these routes and described them in depth. After 1453 the Silk Road was temporarily deviated through the Mediterranean Sea up to Venice, and from Venice it returned back to Xi'an, in China. The cultural exchanges produced along those roads were of great use for human evolution.

7. Eiffel Tower

In 2001 Hawking surprised his readers with a diagram regarding the explosive growth of population [9]. Indeed, after the year 1750, when the industrial revolution started and United Kingdom began to produce iron, the 300 meters tall Eiffel Tower was erected in Paris, France, in two years, between 1887 and 1889. Made of wrought iron, the Tower was twice the height of the Cheops Pyramid in Giza, Egypt, and was shaped as a solid-rigid of equal strength to axial compression in the gravitational field with a well-known exponential equation of governing. Thus, the Eiffel Tower became the second memorial in the world devoted to gravity, but not the last one.

8. Endless Column

Strange but true. From a gravitational perspective, the glory of the Eiffel Tower was short. After less than a half of century, in 1937, the Romanian sculptor Constantin Brancusi created a funeral monument consisting in a succession of 15 cast-iron standing coffins. The monument was devoted to the soldiers who lost their lives during the First World War by defending a metallic

bridge over a mountain river in Romania. The monument was 29.35m in height, ten times shorter than the Eiffel Tower in Paris, and was called the Endless Column by the sculptor, which suggests an unlimited gratitude. In a short time, the interest for the Column quickly increased. However, only in 2001 the author presented to UNESCO in Paris the sculptor's obsession for gravity, and in the subsequent years he discovered that, in fact, the Column consists in a succession of 32 miniatures of the original Babel Tower at the rigorous scale of 1:100. In the act of creation, the sculptor used the automorphic concept based on the mirroring effect discovered by Plato. It was also proved by the Legend of Narcissus, according to which by mirroring gravity disappears and the image becomes free of it. This is why topologically the Column becomes endless, and the dream of liberation from gravity comes true. But this happens topologically only. In reality the access of science to the nature of gravity remains prohibited. In 2005 Stephen Hawking was sorry to accept this bitter truth [10]. The stability tests carried out in the wind tunnel of the Yokohama National University in Japan were successful so that the Column of Brancusi proved to be not only endless but also everlasting. This is why, in the author's view, the Endless Column is the third memorial in the world devoted to gravity. Returning now to Æsopus' fable, it is obvious that the oak represents massive buildings like pyramids and ziggurats, while the reed stands for slender structures like the Endless Column and Eiffel Tower [11-21].

9. Mascrete and reinforcing masonry

Problems of strength never arose in the history of ancient masonry used in nonhomogeneous massive like

ziggurats. The volumes of matter were always high enough to satisfy the requirements of strength. Any intentions of stress concentrations were avoided by the well-known phenomenon of adaptation. Throughout history, the replacement of massive buildings with slender structures reduced the volumes of matter drastically, and stress concentrations around geometric imperfections could no longer be prevented. Generally, the levels of strength of both bricks and mortars started to present interest to structural members and to be put under control. After the year 1867, when Joseph Monier patented the RC in France, solid and hollowed ceramic bricks with high mechanical performances started to be produced, and the lime mortars were gradually replaced with cement ones. Easily and unexpectedly, nothing remained from the old ancient masonry but its colour. A nostalgic, warm one. The author called it simply *mascrete* as an acronym of the two words of origin, masonry and concrete. At the same time, buildings constructed in the interwar period with ancient masonry, with solid porous bricks and pure lime mortars, are still in service in many places. They are in good physical state and worth to be further preserved. In order to upgrade them to the requirements of the new codes for seismic protection, for instance, a method of reinforcing the masonry with polymer grids was patented in Bucharest, Romania, after a careful program of experimental tests [22]. The conceptual base of the method was Lev Landau's Theory of dislocation for which its author was rewarded with the Nobel Prize in the year 1936. The polymer reinforcement used has almost the same strength of 200 MPa, like the ordinary steel used for RC. It is either inserted in the horizontal joints of masonry as reinforcement or applied externally as a

confinement. In both cases the purpose of intervention is to prevent stress concentrations around any geometrical imperfections of masonry structure like the vertical joints. Extensive programs of experimental static and dynamic tests, including anti-terror tests, were successively carried out by the authors in different European Laboratories along a full decade before the beginning of the new millennium. Today the method

is freely applied everywhere, as it is no longer under the protection of the Patent Office. Several experimental results have been selected for the author's short oral presentation planned for the 21st October 2018 at the Workshop in Tehran.

10. Comparative Analysis

For practical purposes of design or research, a comparative table is proposed below. It is based on a large number of references of which only a few have been selected for this paper [23-27]. Regarded separately, each data answers to a certain, precise question. Regarded together, in ensemble, all data reflect the great difference between the past and present time. This is why the future seems so unpredictable.

11. Durability of Interventions

Durability is a probabilistic concept defined by the function:

$$r = \int_0^{\infty} F(t) dt \quad (1)$$

where, in the general case, reliability assumes the expression

$$F(t) = e^{-\int_0^t \lambda(r) dr} \quad (2)$$

where $\lambda(r)$ is the function of risk. The analysis is based on two laws of happening established by Georges-Louis Leclerc:

Something and something and something...

Something or something or something...

and two principles:

Safe life, and Fail safe

The results of the analysis are of great practical interest [28].

PHYSICAL AND MECHANICAL CHARACTERISTICS	MASONRY	MASCRETE
1. Type of material	Solid, porous	Consolid, ceramic
2. Type of mortar	Line-durable	Consistent - brittle
3. Specific weight	18 kN/m ³	20 kN/m ³
4. Age in years	>4,000	<200
5. Young modulus of elasticity - average values	0.8 GPa	8 GPa
6. Coefficient of thermal expansion	4x10 ⁻⁶ /°C	10x10 ⁻⁶ /°C
7. Thermal isolation k in kcal/m ² /°C	1.82	2.10
8. Acoustic isolation in dB	35	48
9. Strength ratio: masonry/brick < 1	Yes	No
10. Dependence on gravity in service	Yes	No
11. Direction of mechanical properties	Orthotropic	Quasi-isotropic
12. Stability for seismic actions	No	Yes
13. Contents of included energy	9.20 J/m ²	7.4 J/m ²
14. Water content in mortar	Dry	Wet
15. Mechanical behavior	Ductile	Brittle
16. Compatibility with concrete and R.C.	No	Yes
17. Type of compatible reinforcement	Non-metallic	Metallic, Steel
18. Type of reinforcing	Passive only	Active and passive
19. Stress transfer from mortar to reinforcement	Anchoring effect (a)	Clamping effect (c)
20. Vertical joints	active	passive
21. Monolithic	No	Yes
22. Stability for prestressing	No	Yes
23. Stability to reinforcing with fibres	No	Yes
24. Stability to perforation	No	Yes
25. Sandwich effect	Yes	No
26. Vack effect	Yes	No
27. Reversibility	Yes	No
28. Recovering the bricks by demolition	Yes	No
29. If it hides a professional secret?	Yes	No
30. Compatibility with wood	Yes	No
31. Compatibility with masonry in monuments	Yes	No
32. Response displayed on the shaking table	Mortar yields	Brick cracks
33. Flammability, according to ISO 13623:2011	High	Low
34. Cost of the non-invasive masonry treatment	145 Euro/m ²	285 Euro/m ²
35. Resistance to immersion in water	Low	High
36. Resistance to freezing	Low	High
37. Resistance to polluted air and acid rain	Low	High

12. Conclusion

Any intervention should be typologically and existentially compatible with the considered building chosen for therapy. Typology means structure or massive, while existential means gravitationally alive or dead. Gravity is one of the four fundamental forces of the universe. It is the weakest force by far, but it is the most permanent and penetrant force. It is the only force of unknown nature, even if it is felt by all the bones of *homo gravitas*. Every intervention should start from the cause of damage or failure, and not from its effect.

The foundation of the building should always be the first considered for examination with maximum priority. The fact that the foundation is not seen does not mean it does not exist. The estimated durability of the intervention may be a good criterion for quality control. The decision of intervention should be lucidly taken by one's own individual mind, not collectively. It should be taken without any external influence if possible, but it should always be checked by one's own heart. Finally, don't forget Heraclitus, time is one-way flow. It never returns.

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Flexible Bonding in Protection of Masonry Structures

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Abstract

In this paper, innovative intervention methods for retrofitting and protection of masonry structures against earthquakes are presented. They are based on polyurethane flexible joints allowing transfer loads and high deformations. Presented examples of laboratory and in situ tests, carried out on flexible joints and on flexible adhesives bonding composite strengthening, manifest efficiency of the proposed intervention methods, also in application on historical structures. Requirements of using monitoring in the historical structures and using of it for control of intervention systems are discussed. Critical view on using of innovative methods in retrofitting of historical structures is also presented.

Keywords: Weak masonry substrates, Reduction of stress concentration, Flexible bonding, Repair and strengthening, Protection against earthquake

1. Introduction

1.1. Principles of intervention in historical structures

Requirements of structural intervention in historical

structures are current all over the world. Many of these valuable structures are adobe [1] or masonry (typically made of brick or stone) constructions, which have not been designed to withstand extreme loads induced by earthquakes (Figure 1). These loads induce forces acting against gravity loads (horizontal dynamic forces) and cause damages such as cracking or even structural failures. The use of “traditional” and new materials and repair/strengthening systems for retrofitting of masonry historical structures has been analyzed and tested for last decades not only in laboratories but also in situ on real structures. In many cases, improper structural interventions with new incompatible materials and systems (e.g. Reinforced Concrete – RC) were verified by next earthquakes with catastrophic consequences to heritage constructions. However, some of them allowed surviving historical structures starting to be a part of them treated as historical nowadays.

Historical structures of critical importance and uniqueness (monuments) and also other ones presenting cultural significance have to be protected for future generations. To avoid improper interventions harming cultural heritage, the international regulations [2-5]



Figures 1. Bam Citadel: 1a. before and 1b. after the earthquake [1].

have to be followed. These documents provide basic principles for retrofit measures, which have to satisfy basic principles such as durability in time, compatibility and reversibility. Very important is the minimal intervention rule, in which repair or strengthening techniques improve resisting mechanisms of the structure and structural behavior at the level judged as satisfactory in each case study, causing only the strict minimum necessary significant alteration on the features. Following them, the use of new extensively invasive and inappropriate techniques for rehabilitation and retrofitting of historical structures has to be avoided. It should be pointed that monuments with their uniqueness are not allowed to be test cases for new materials and intervention techniques, thus the retrofitting systems applied on them have to manifest confirmed usefulness and harmlessness in long time usage. However, other ordinary historical masonry constructions (city centers e.g. Bhaktapur in Nepal), presenting cultural significance but not protected as efficiently as the monuments, need cost-efficient retrofitting solutions aimed at mitigating their vulnerabilities against earthquakes. In this perspective recent advances in scientific knowledge,

developing innovative systems improving the structural stability of historical structures (considering compatibility and reversibility), are in scope of engineers' and conservators' interest.

Many of them were applied in practice and have been manifested their efficiency till now.

1.2. Emergency intervention

Safety of damaged historical buildings (after an earthquake) is the economical and social problem causing often non-use of these buildings for many months or years. A big part of existing residential and religious buildings in seismic areas are masonry buildings situated in the historical centres of cities and villages. In case of seismic actions only a part of them undergoes collapse or failure when serious damages unable to bear service loads anymore. Many of them are cracked but still able to support the normal service load, but unable to support strong seismic actions. In these cases authorities or engineers generally declare them as not usable because of the aftershocks risk. This fact causes the problem of providing provisional home to a big part of the inhabitant of the city centres and of the villages affected by the



Figures 2. Damages of the church of San Francesco in Mirandola (top) and of the church of D.O.M. in San Felice sul Panaro (bottom), caused by the first shock and the aftershocks [8].

earthquake. This is not only an economical problem but also a social problem, because nearly all the relationships among the citizens are interrupted, as shops and workplaces are closed and normal life in general is hindered strongly for a great number of persons.

Many examples of historical masonry structures which survived first shock of earthquakes with minor damages but being weakened by them collapsed during aftershocks are reported. Such cases were lastly reported in the case of the Emilia Romagna Earthquake [6-7] which hit Italy in May 2012. The 5.9 Mw earthquake took place on 20th May 2012 and was followed by 5.9 Mw aftershock on 29th May 2012. The damage of the church of San Francesco in Mirandola and the church of D.O.M. in San Felice sul Panaro after the first shock and after the aftershocks is shown in Figure 2. The period of time before an aftershock, during which suitable temporary repairs could be made, can be rather very short (e.g. in the case of the Emilia Ro-



Figures 3. Damaged heritage structures in Kathmandu (Nepal), vulnerable to strong aftershocks.

magna Earthquake), thus quickly using repair (protection) methods are required. Such intervention also makes sense in case of valuable cultural heritage structures which are difficult to reconstruct after catastrophic collapse (Figure 3). It is believed that it is better to protect than to repair or reconstruct also from the economical point of view. Short time before an aftershock does not allow for choosing a final repair strategy and waiting for the authorities' consent about application of strengthening methods. In most cases, earthquakes devastate large areas and many structures and there is no time to think about the most suitable repair for a particular structure. However, there could be enough time to engage civil engineers to take care of damaged buildings using temporary measures without engaging authorities, as they need more time for analysis. The temporarily protected structures can be deeply analyzed by teams of experts, to choose the most adequate repair or strengthening methods.

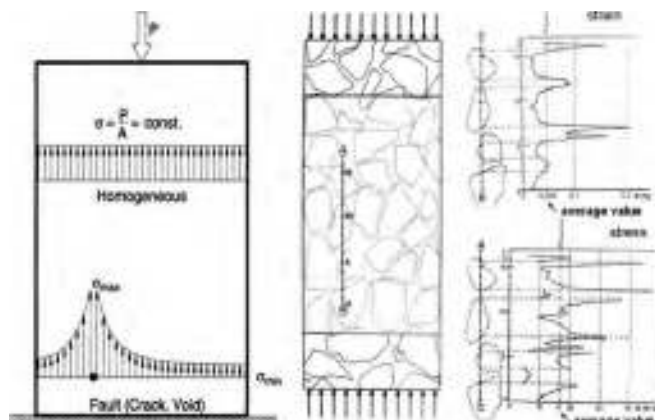


Figure 4. Stress concentrations generated in stiff brittle masonry under loads (deformations) [9].

1.3. Stress concentrations in masonry structures and possibility of their reduction

Brick and stone masonries, especially historical ones, manifest usually low strength caused by weakness of the used construction materials, ageing or environmental deterioration as well as damaging actions, e.g. earthquakes. Masonries undergo often damages during dynamic excitation, because they are made of stiff and brittle materials of rather heavy mass. Even low deformations generate stress concentrations in stiff masonry (Figure 4), which cause micro-crack appearance and finally main cracks are being created, when the strength of weak materials is overcome. A cracked masonry structure undergoes stress redistribution after damage, when transferring loads with zero stress in the cracks location. Minimal optimum intervention requires using of protection, repair or strengthening technologies, which would not be neither introduce or generate additional forces nor cause stress concentrations in weakened historical substance. Moreover, newly applicable intervention materials should fulfil requirements of

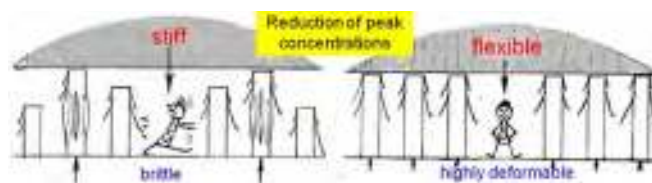


Figure 5. Scheme of difference in stiff and flexible behavior [10].

mechanical compatibility, to avoid creating new damage. For damaged heritage buildings, one important consideration in making protection and strengthening of massive masonries in seismic areas is improving of earthquake resistance of structures with strengthening systems carrying additional loads, increasing structural ductility, but not increasing structural mass (structure should be as light as possible) nor stiffness.

These conditions can be realized by application of new flexible bonding systems or flexible adhesives using in fastening of composite strengthening. Flexible protection, repair or strengthening systems use special family of neutral polyurethane materials of high durability, confirmed by various laboratory and in-situ tests. They are characterized by hyperelastic properties with about three orders lower stiffness than masonry and deformability withstanding strains over 100%. Flexible joints or adhesives made these polyurethanes are able transferring high loads and high deformation simultaneously and dissipating huge amount of strain energy during dynamic or cyclic action (are resistant to fatigue tests). The main advantage of flexible bonding is ability to reducing of stress concentrations and redistributing them over a large bonding area (Figure 5). This phenomenon allows for transferring even high loads by weak masonry substrates (historical) because they are protected against stress concentration, larger than the weak substrate strength.

1.4. Innovative protection, repair and strengthening methods

In this work, examples of using flexible bonding in protection, repair and strengthening of weak masonries are presented, basing on laboratory and in-situ tests. To explain ability and advantage of the new flexible systems, simple theoretical background is shown with comparison of tests results of stiff and flexible bonding work. These results are presented also with the use of Digital Image Correlation (DIC). Examples manifesting efficiency of flexible bonding are grouped for repair of cracked masonries and strengthening of them using composite materials. Short discussion of using of composite materials with stiff and flexible adhesives is presented. It is also proposed a new philosophy of protection of historical masonry structures against earthquakes and temporary quick protection of them against aftershocks, when damaged by a main shock of an earthquake. Flexible bonding using special polyurethanes can help effectively in protection, repair or strengthening of historical structures, without causing of additional damages in heritage structures. Moreover, the flexible systems can improve the structural ability of heritage objects to withstand damaging earthquake actions. The proposed flexible systems can protect the damaged historical structure and human life against catastrophic results during aftershocks actions and give time authorities to make the right decision on repair or restoration of valuable historical structure. However, it is important to underline that application of innovative systems in damaged heritage structures should be preceded by deep study and analysis of retrofitted heritage structures and interactions of them with the proposed intervention, following ISCARSAH

recommendations. Moreover, control of efficiency and work of applied innovative solutions as well as of behaviour of the retrofitted heritage structure should be monitored during their exploitation. As it was mentioned previously, innovative methods are not allowed to be tested on monuments, except situations when only they allow for surviving of the valuable heritage against catastrophic consequences (e.g. collapse).

2. Polyurethane flexible joints

2.1. About innovation

Polyurethane Flexible Joints (PFJ) is an innovative system, developed at Cracow University of Technology (CUT) in Poland by a group of scientists. The PFJ consists of a group of various ideas, which have been developed since 2003 and are protected now by patents and knowhow for Cracow University of Technology (Poland). It is commercialized now by the spin-off company FlexAndRobust Systems Ltd (FAR) by virtue of a full and exclusive license agreement, in cooperation with other industrial partners. FAR was set by innovators – members of the application group. The PFJ develops still, covering gradually various areas of construction industry, being applied in masonry, concrete, reinforce concrete, steel, timber structures and composite strengthening. Over 130 papers have been published in international and Polish journals and on conferences since 2004. The PFJ were also applied in protection of heritage structures in Poland.

2.2. Characteristic of new structural connections in civil engineering

Lack of suitable structural solutions in civil engineering,

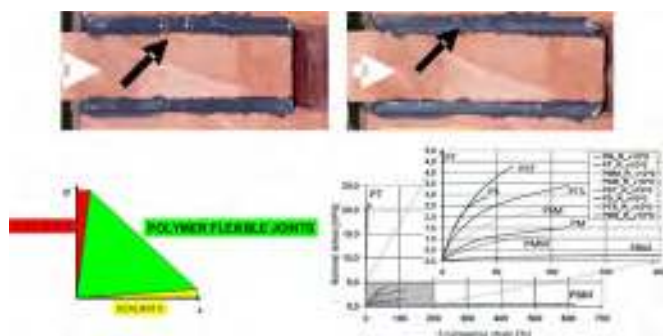


Figure 6. Polymer Flexible Joints, made of special polyurethanes, capable to carry high loads and high deformations simultaneously in masonry structures, made of special polyurethanes characterized by hyperelastic (non-linear) behaviour.

related to creation of structural connectors based on bonding, having ability to transfer loads and high deformation (e.g. during earthquake) simultaneously laid as background for the need of this innovation. Majority of civil engineering structures (also historical ones) are masonries or concrete with their brittle properties of structural materials. Before the PFJ introduction, connectors of brittle structural members were belonging to opposite areas: stiff joints (capable to carry high loads but unable to carry high deformations) and sealants (capable to carry high deformations but unable to carry high loads). The gap between them was empty in the case of construction industry, whereas in motor and marine industries this gap was fulfilled by flexible connectors, able to transfer high deformations under static and dynamic loads. Following those branches, a new sort of structural joints was introduced in construction industry, named Polymer Flexible Joints (Figure 6). The main advantages of the PFJ come from hyperelastic (non-linear) characteristic of special polyurethanes forming flexible joints. They are capable to carry high loads and high deformations simultaneously.



Figures 7. Cracked historical masonry structures with not stable foundation (Venice) and located in seismic areas (Nepal).

They also allow for reducing of stress concentrations and redistributing them over large bonding area.

2.3. Drawbacks of stiff brittle connections undergoing high deformations

Civil engineering structures made of brittle materials (concrete, masonry), located in seismic and settlement areas or places with not stable foundation (e.g. Venice – Figure 7), on active slopes or all places where dynamic and thermal loads occur, are vulnerable to high deformations and easily undergo damage. The need of transferring loads and deformations simultaneously by structures in seismic areas is obvious, but the need of reducing stress concentration and redistributing them over large bonding area is also crucial. It is known from many researches that under the peak of stress concentration brittle materials reach easily their limit of elasticity and start



Figures 8. Cracked historical masonry after improper repair using stiff cement mortar.

forming micro-cracks locally (Figure 4), which link into a main crack after load increase. The stiffer joint between two brittle structural elements is, the higher peaks of stress concentrations are, responsible for micro cracks formation and thus weakening of brittle materials. Reduction of stress concentrations assures highly deformable flexible polyurethanes. This idea was schematically presented in Figure 5.

Both presented aspects are manifested together in the case of repair cracks in brittle structural elements made of concrete or masonry. In the surrounding of the crack, a weakened zone remains full of micro-cracks. When the crack divides a structural element into two parts, they work separately in the new stress equilibrium and static balance being the consequence of stress redistribution. Separated parts of the cracked structure can draw apart e.g. under cyclic repeated settlements caused by fluctuations of a water level, temperature changes and vibrations (seismic or ambient), of even not very high intensity. Typically, cracks are repaired using stiff and brittle mineral or epoxy (not allowed in retrofitting of historical structures) grouts, which do not improve significantly the structural capacity (damage energy), because they introduce stress concentrations again and low deformability (disadvantageous in seismic areas, where materials of high ductility are required). An example is the case of a

cracked historical masonry building damaged by settlement, which was repaired by injecting of cracks using stiff and strong cement mortar. Large deformations and high stiffness of the repair joint generated high stress concentrations in weak and brittle masonry, caused appearance of new cracks in surrounding of the repair (Figure 8).

3. Repair and protection of cracked masonries by injection

3.1. Work methodology of flexible bond (injection)

The innovative bonding (repair) method uses flexible joints made of special highly deformable (injectable) polyurethanes of reasonable low stiffness, instead of traditionally using stiff and brittle bonding materials of low deformability (epoxy resin, lime and cement mortars). Flexible joints are constructed from polyurethanes of various stiffness and deformability, chosen properly in the PFJ to assure compatible work with various substrates under differential loading conditions. Properties of the PFJ can be designed for various requirements. There have not been any structural joints of presented properties in construction industry till now, what was confirmed by patents and recognition in scientific publications.

This kind of highly deformable material reduces stress concentrations and more evenly redistribute them along the whole bonding surface (Figure 5). The new kind of flexible joint introduces greater tensile and shear resistance, deformability and ductility and thus greater bearing capacity (damage energy) in the bonded brittle structural elements, also in the post

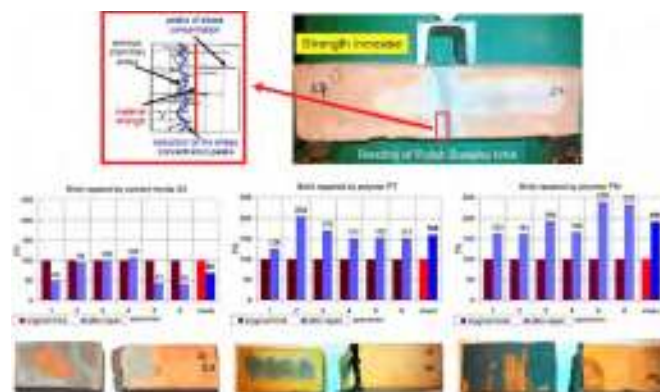


Figure 9. Strengthening efficiency of repair joints made of cement mortar (ZA) and two polyurethanes PT and PM, tested in 4-point bending, compared to original strength of the bricks (100%) [11].

failure deformation zone - making the structure more safety. The most popular application of the PFJ are: repair of cracks by injection or filling structural gaps to make flexible hinges (using polyurethanes PM and PT) – system named as polyurethane flexible joints (PUFJ). Polyurethane PM is characterized by low Young's modulus $E = 4 \text{ MPa}$ and high deformability $\varepsilon = 120\%$ and elasto-visco-plastic mechanical behavior, providing a high amount of damping and ductility. Polyurethane PT is stiffer and less deformable ($E = 600 \text{ MPa}$ and $\varepsilon = 10\%$).

3.2. Comparison of repair efficiency

Comparison of bonding efficiency of PUFJ in the aspect of ultimate loads carried by repair joints, made of various materials of low and high deformability, was presented using specimens made of brick units. The tested units were first broken in a bending test and next repaired by bonding in the place of crack. The results of the tests on specimens with various bonding materials

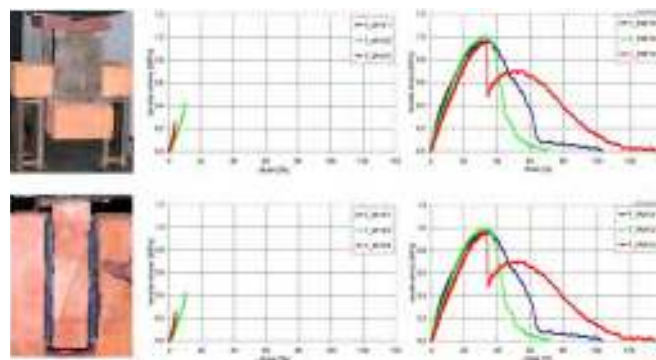


Figure 10. Strengthening efficiency of repair joints made of cement mortar (ZA) and two polyurethanes PT and PM, tested in 4-point bending, compared to original strength of the bricks (100%) [12].

joining disrupted bricks (simulating cracks repair) indicated that repair of cracks in masonries by the use of cement mortar or epoxy resin (stiff materials of low deformability) is less effective (taking into consideration the load level) than using of flexible polyurethanes of high deformability. Example results for the joints made of cement mortar (ZA) and two polyurethanes PT and PM are presented in Figure 9 [11].

3.3. Comparison of new joint efficiency

To compare brittle and stiff joints, simple tension and shear tests were carried out on inorganic mortar grout and deformable polyurethane joints. Specimens of small dimension made of typical Polish solid clay bricks (Bonarka) were used. The bed joints between brick elements were made of two kinds of materials of 10 mm thickness in each tested case. As the stiff brittle joint, lime-cement mortar of class M7 was used. As the deformable joint, polyurethane PM was applied. In each test, three specimens with mortar – marked (M) and

with polyurethane PM – marked (PM) were investigated. Results in form of stress-strain curves (in comparable scale), obtained during tension and shear tests, are presented in Figure 10. Tension and shear tests confirm that joints made of polyurethane PM have much higher bearing capacity than joints made of lime-cement mortar [12].

3.4. Safety approach of using PUFJ in cracked masonry structures

Work scheme of the flexible joint constructed in the crack of a damaged masonry structure are presented on the basis of the example shown in Figure 11. The polyurethane filling the crack is selected so that its strength is less than the strength of the masonry wall ($f_j = 0.67 f_s$), it also has much greater deformability than construction material ($\epsilon_{us} = 0.35\%$, $\epsilon_{uj} = 100\%$). The description of the example is given below, with a discussion of the next stages of work.

Under operating load, the principal tensile stress $\sigma_{\perp}$ in the masonry is less than its tensile strength, and the energy supplied to the system by the applied load

(equivalent to the area under the curve $\sigma_{\perp}-\epsilon$) is absorbed by the elastic deformation of the structure as the energy of wall deformation - (Step 1) in Figure 11. Under the limit load, the principal tensile stress reaches the value of the tensile strength of the wall, and the energy supplied is absorbed by the structure as the total energy of wall deformation.

When the amount of energy supplied increases further, a crack appears and in its place the principal tensile stress is zero (Step 2).

After filling the crack with polyurethane mass, the stress is still zero (Step 3).

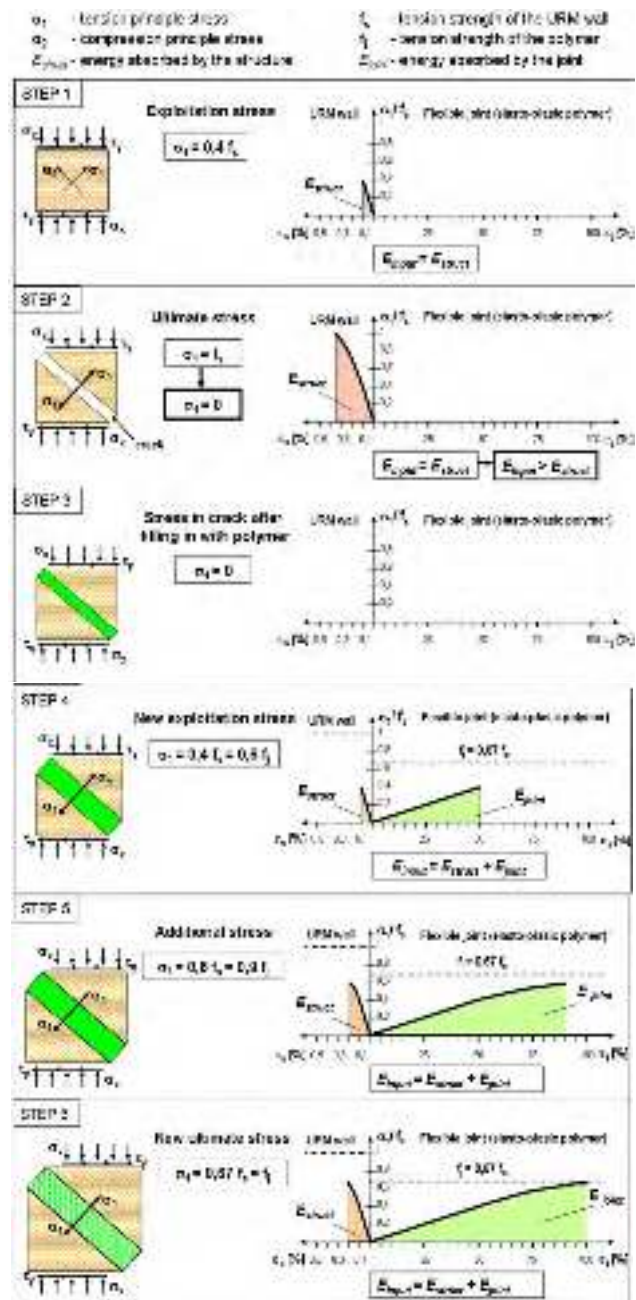


Figure 11. Work scheme of the flexible joint constructed in the crack of a damaged masonry structure.

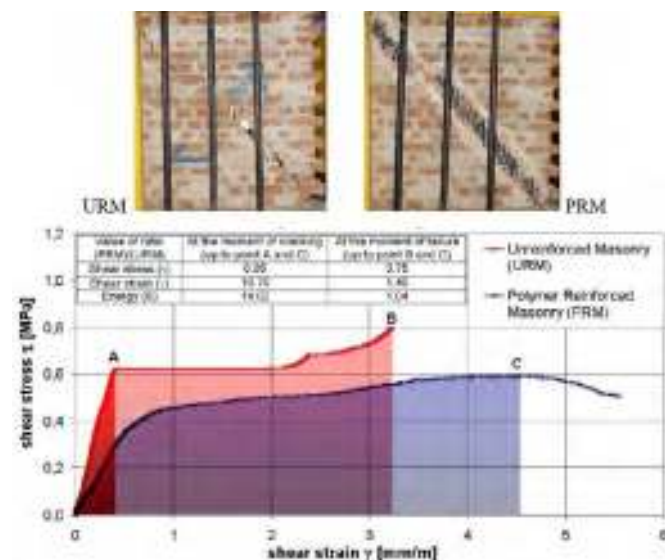


Figure 12. Comparison of the results for the unreinforced (URM) and the polyurethane reinforced masonry (PRM) specimens [11].

Under the influence of the new operational load, the energy supplied to the system is simultaneously absorbed by the elastic deformation of the wall and the newly developed flexible joint (sum of areas under the σ - ϵ curves draw for the wall and for the joint) (Step 4). The additional energy supplied to the system (from the additional load) is simultaneously absorbed by the elastic deformation of the wall and the non-linear deformation of the flexible joint (Step 5). Under the influence of the new limit load, the principal tensile stress reaches the value of the tensile strength of the polyurethane and the joint is destroyed, thus protecting the historic structure from the occurrence of additional damage (Step 6). It should be noted that the amount of energy needed to destroy the joint in the given example is much greater than the maximum deformation energy that the wall itself is able to carry. This

approach was examined during laboratory research, where efficiency of highly deformable repair material was checked. A shear test on a cracked unreinforced masonry wall (URM), next repaired using the polyurethane PM and tested again (PRM), was carried out. Results are presented in Figure 12 after [11].

The wall pre-compressed vertically was subjected to horizontal shear force in one cycle. The diagonally cracked masonry wall was strengthened by polyurethane PM injection, and 24 hours later was again forced horizontally up to failure. The final damage occurred only in the polyurethane joint (important in the case of heritage masonries). Cracked masonry wall, repaired by injection using highly deformable polyurethane PM, restored the original masonry strength up to 95% (Figure 12), but additionally increased ultimate shear strain over 10 times and energy dissipation capacity 14 times. A huge amount of ductility was introduced in the damaged masonry structure in comparison to the original one (it is important especially in seismic areas), what increased construction safety significantly.

3.5. Efficiency of PUFJ tested masonry building up to failure

Efficiency of polyurethane PM joint was examined on small masonry building during in situ dynamic tests up to failure. The original structure was damaged by a caterpillar. Ultimate dynamic forces were generated in the corner of the building at the roof level. The structure was about to collapse after the action and had to be rectified what caused appearance of new cracks, looks like after a strong earthquake. The cracked building was repaired by injection using highly deformable polyurethane PM. Dynamic destructive test



Figures 13. Dynamic in situ tests up to failure on small cracked masonry building, repaired by flexible joints made of polyurethane PM.

using caterpillar showed again that the strength of the highly deformable repair joint can be higher than the original masonry. After the huge hit of the caterpillar a new crack appeared in a new form, different from the primary damage. Moreover during unloading of the structure, the polyurethane joint caused closing of the newly appeared cracks. The disrupted part of the wall of triangle shape (protected by flexible joint) was moving during hits, like on the pair of rubber braces. The destructive test showed that the polyurethane reduces stress concentrations and introduces the huge amount of capacity into the cracked masonry structure absorbing the input energy. New damages went only through new masonry areas, not through the polyurethane bonded cracks. Finally, the last huge hit collapsed the repaired masonry structure but the polyurethane PM kept fast together pieces of wall

during the damage process (Figure 13).

The tests showed effectiveness and advantages in the use of highly deformable bonding polyurethanes as the repair material of damaged masonries in cases of static and dynamic loads. The destructive tests showed that polyurethane joints reduce stress concentrations and introduce the huge amount of capacity to the cracked masonry structure to absorb the input energy. The flexible bonded masonry could be able to survive an earthquake and assure construction safety better than an original masonry. The presented innovative bonding technology is also able protecting architectural value of structures because it assures the rule of minimum intervention (is compatible with masonries structures from the mechanical point of view) without significant intervention in elevation of buildings and can be easily hidden under conservation materials fitted to the surroundings view, making durable joint of aesthetic external appearance (Figure 14).

3.6. Real application of PUFJ on weak cracked masonry building

The PUFJ was tested on a masonry family building, constructed from spider-web rubble walls of poor quality. The walls consisted from sand-stones joined with weak lime mortar. Damages in the building appeared in form of cracked wall and were caused by settlement after a flood. The cracks were filled in with the flexible polyurethane PM (Figure 15). Effectiveness of the flexible joint was examined during the window exchange. The triangle wall fragment was suspending only on the newly constructed flexible joint for the period of several hours. There were no fissures on a wall plaster and on gypsum markers placed on the joint (as a simple



Figures 14. Polyurethane PM easily hidden under conservation materials fitted to the surroundings view, making durable joint of aesthetic external appearance (can be used in protection of frescos).



Figures 15. Cracks of masonry family building, constructed from spider-web rubble walls of poor quality, repaired by polyurethane PM flexible joint.

monitoring system) as well as no cracks have been observed or fissures on structure till now (for 12 years). This case confirms that flexible joints equalize stresses in brittle materials and protects cracked masonries of the weakened structure against appearing of new damages, during micro-movements occurrence of the separated building parts.

3.7. PUFJ applied for protection of cultural heritage

The PUFJ can help in efficient protection of cracked heritage masonry buildings in seismic areas. Valuable



Figures 16. Protection (by prefabricated and constructed on side PUFJ) of heritage sculptures on palaces roofs in Warsaw against traffic vibrations destructing them.

damaged structures are not allowed to be reconstructed (Figure 3 – cracked heritage structures in Kathmandu after the Nepal Earthquake 2015), thus the best way of assuring construction safety and recovering aesthetic view seems to be the use of PUFJ with the proper external cover (Figure 14).

High effectiveness of PUFJ was examined also on real structures during fastening of heritage sculptures on palaces roofs in Warsaw. The aim of flexible bonding was protection of stone figures against traffic vibrations destructing them. The prefabricated PUFJ and constructed on side have been working properly for 8 years (Figure 16).

Valuable cultural heritage museum exhibits have to be protected against damage during earthquakes, when are located in buildings in seismic areas or are endangered by dynamic actions (e.g. traffic vibrations). Even small shocks (not harmful to buildings) can demolish historical goods thus proper anti-vibration protections should be used. An example of such protection is presented in Figure 17, where the prefabricated PUFJ damps vibrations at the bottom of museum exhibits in National Museum in Warsaw (Poland).



Figures 17. PUFJ damping traffic vibrations at the bottom of a column (Paulos Cathedral from 8th century rescued in ancient Pachoras) in National Museum in Warsaw (Poland).

3.8. Durability aspect of PUFJ

Because of environmental and dynamic influences, PUFJ joints have to be resistant to cyclic (fatigue) loads. This ability was confirmed in research on a glass-timber beam with PUFJ interface, which was tested in the four point bending cyclic test, under over 100,000 cycles. The maximum force level of cyclic loading was estimated based on the “reference” beam, assuming 50% of the maximum load causing the initial crack in the glass web. The applied cyclic load was ranging 0.7 - 2.4 kN with the frequency of 0.62 Hz. In the absence of evidence of destruction of the components after 100,000 cycles, quasi-static load was applied to the beam up to failure. The cyclically tested beam behaved in very stable and linear-elastic manner. The rigidity of the beam during the test was only slightly reduced, which showed an increase in deflection in the cycle of 100,000 by about 2% compared to the deflection in the first cycle. Both beams,

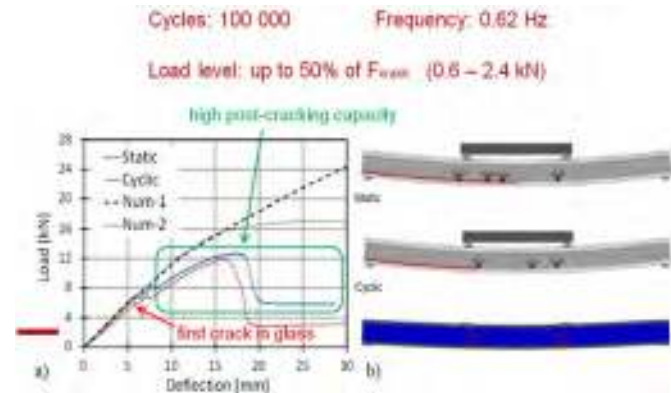


Figure 18. Relationship between load and deflection of the investigated beams, compared with numerical calculations (a), the failure schemes of the beams (static and cyclic) and calculated cracks locations of the glass web (b) – after [13].

subjected to cyclic loads and quasi-static load, exhibited as well significant post-breakage load-bearing capacity (load value between first crack appearance and element failure) ranging up to 200% of the load value causing the first crack in glass web and ductile work (Figure 18) [13].

Durability and high efficiency of PUFJ were confirmed during 8 years exploitation of a cracked concrete airfield slab in the Cracow-Balice Airport (Figure 19). The repair flexible joint made of polyurethane PM



Figures 19. The repair flexible joint made of polyurethane PM, resistant to environmental and chemical factors acting on the joints at the cracked concrete airfield slab in the Cracow-Balice Airport.

was able to withstand all environmental and chemical factors acting on the joints during summers (up to $+40^{\circ}\text{C}$) and winters (below -20°C) in Poland. The extreme cyclic loads and high deformations (coming from main gears of airplanes) were also carried by the flexible joints without any damage. The ability of transferring high loads and high deformations by PUFJ (made of polyurethane PM) was confirmed during cyclic laboratory tests in real scale. The hysteresis loop manifests good damping properties of the flexible joint [14].

4. Monitoring of structural behavior before, during and after retrofitting (control of innovative interventions)

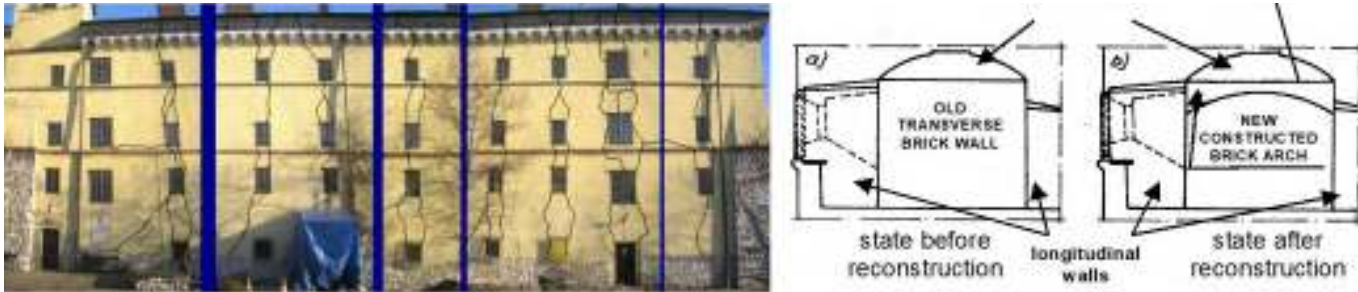
4.1. Requirements of monitoring

Somebody working as an expert on evaluation and retrofitting of historical structures needs proper tools that can help in giving an opinion about reasons of damages visible on the damaged structure and then an intervention decision recommending proper repair or strengthening methods. It should be pointed, that deep investigations of various aspects has to be done (consulted with experts of various specialization) in the decision process. In most cases related to the damaged historical structures, experts have to work as detectives, thus needs accurate information about a structural behaviour of a investigated structure, working under environmental, exploitation and accidental conditions. This information is required not only before retrofitting intervention but also during it as well as after finishing of it. There are many monitoring systems but in most cases simple and effective

methods giving enough information are wanted. Effectiveness of two of them are presented below (settlement measuring by a leveller device and cracks width changes measuring by a mechanical extensometer).

4.2. Long term monitoring in historical building before and during retrofitting

An old masonry building from the beginning of the XVII century (Cracow in Poland) manifested damages of the West Wing caused by the structural reconstruction in the sixties of the XX century. Changes consisted in removal of some transverse bearing walls on two upper floors (protected against settlement) and the appeared large openings were covered with flat brick arches negligibly executed, which supported cloister vaults (Figure 20). The introduced changes resulted in decreasing transverse stiffness of the building and huge uneven settlement of the West Wing. Appeared damages were covered by plaster but after 40 years of exploitation cracks started to be visible on the elevation. They were caused by introduced horizontal forces generated by arches construction and resulted in strutting of the longitudinal bearing walls, increasing thus appeared damages. Cracks and fissures went through vaults, arches and walls of the building. After 4 years of settlement monitoring (1996-99) no changes were observed, and small settlement was noticed (1999-2003) after deep excavations in a constructing site near the West Wing and by drought in summer 2003 (Figure 21). In the same time, the cracks width monitoring indicated that damages in vaults and arches are not stable, because manifest a trend of crack width increasing (fluctuating with seasonal thermal changes were clearly observed) -



Figures 20. Damages at the West Wing elevation and schematic visualization of the harmful structural reconstruction of the building.

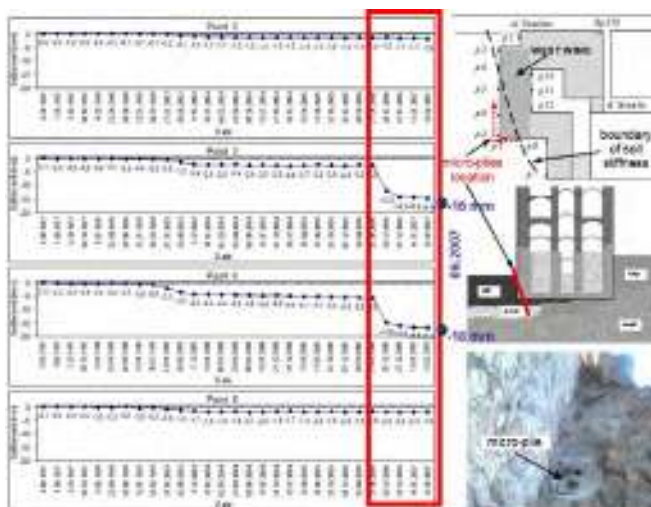
Figure 22. Dynamism of this process threatened with loss of stability of arches and vaults, supported by strutting longitudinal walls, thus strengthening of arches was projected in the year 1999 and constructed in the West Wing in the year 2002.

It was required to join the longitudinal bearing walls and cracked arches to allow them carry additional horizontal loads from thermal wall movements and from potential loads generating possible new damages. The applied retrofitting method had not introducing additional forces in the historical structure but protecting the object against additional factors, which could affect the existing rather stabile state of the structure (ISCARSAH rule of minimal intervention). Minimal intervention in the building elevation and structure was an essential factor. The cracked arches were strengthened using CFRP laminates S512 (Figure 23). Details of material properties and construction process were presented in [15]. Applied strengthening system was one of the first solutions of this type applied to a historical masonry structure in Poland. The strengthening based on CFRP laminates, anchored in the brick wall by the use of glued in steel anchors, permitted for a quick, effective, and relatively inexpensive protection of the damaged part of building.

Application of CFRP laminates permitted also to include the strengthening into full co-operation without introducing of pre-stressing, which could influences negatively the damaged object being in a relative state of balance.

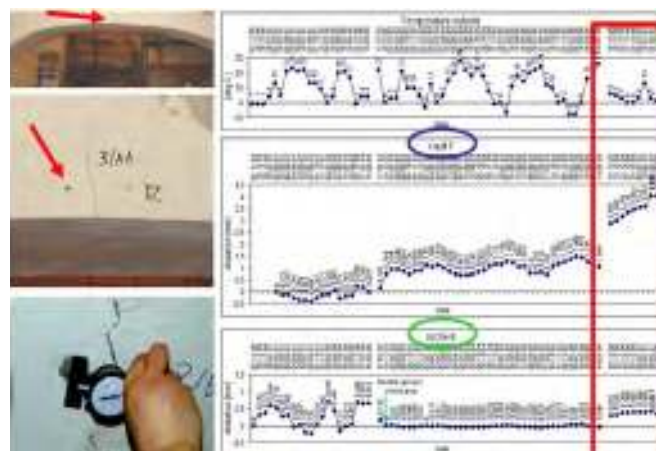
Over four year's monitoring of changes in the width of fissures in repaired arches showed that the strengthening worked properly according to the designers' expectations. There was not observed any increase of fissures width at arches (see measurements for the arch E in Figure 22) after increase of settlement in the period 1999-2006 (Figure 21), when such increase was observed at vaults (see measurements for the vault F in Figure 22) caused simultaneously by settlement and season temperature changes.

Finishing the expert process in July 2006 (after over nine years of monitoring and strengthening of structure with minimal intervention), repair recommendations were given to the investor and the building owner to be realized in following order: global stiffening of whole building (first), stiffening of weak soil supported the West Wing foundation (second). This order of repair was obligatory because of weakness of the West Wing structure and risk of stability loss.



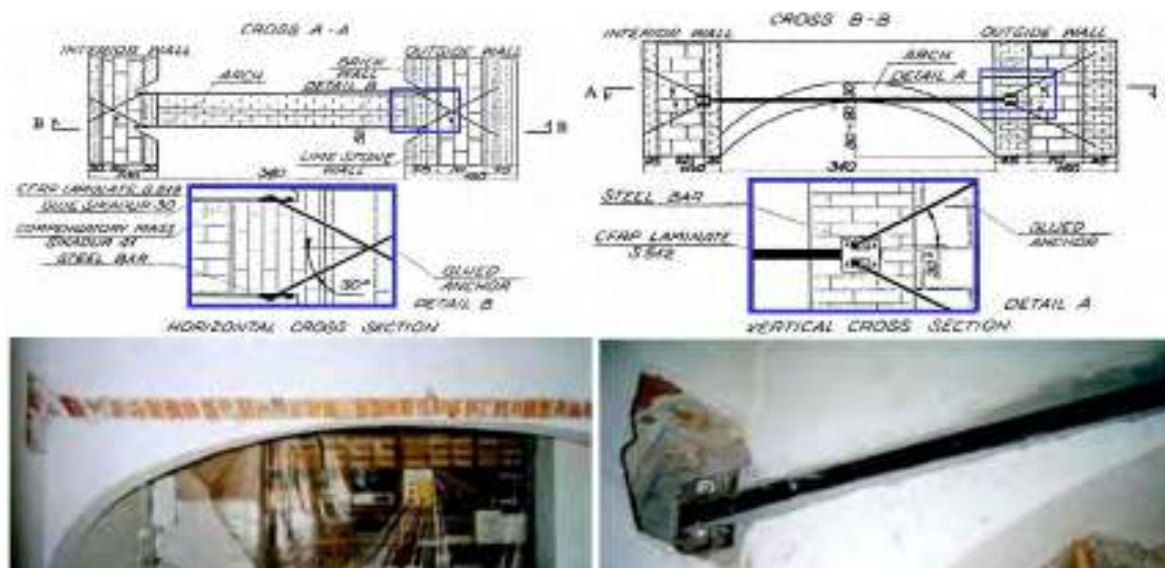
Figures 21. Monitoring results of settlement of the West Wing and location of micro-piles.

Unfortunately, the other way of repair was chosen for realization by the administrative decision body, leaving out the given expert opinion. They decided to



Figures 22. Monitoring results of cracks width changes at the vault F and the arch E.

strengthen first foundation using dozens of micro-piles going through limestone foundations and next to stiff the whole structure. This decision caused dangerous



Figures 23. Strengthening of arches in the West Wing using CFRP laminates with epoxy adhesive.

consequences. Constructing of micro-piles (Figure 21) was started in December 2006, causing vibration of the whole structure (by drilling of limestone walls) and resulting in liquefaction of the weak soil under foundation. Application of micro-piles weakened the peat soil layer causing rapid settlement of the building part of value 10 mm in two days (Figure 21– red frame).

The settlement started stabilizing up to value of 18 mm after six month after stopping of palling. Because of this movement, new cracks appeared on vaults and walls up to 2.5 - 3.0 mm width and the old ones opened increasing they width of similar value (see vault F in Figure 22 – red frame).

Additionally, there appeared new fissures at the arches, indicating that the support balance of the strengthened arches was upset, generating additional deformation and stress in the CFRP laminates (see the arch E in Figure 22– red frame) and also in the anchors (Figure 24).

The presented results of continues monitoring informed about structural behaviour of the building and confirmed that the applied intervention using composite strengthening survived the structure from catastrophic consequences. Finally, geotechnical works were stopped and the whole structure was stiffened in the whole building solid.

The presented above case study clearly indicates that long term monitoring using simple measuring devices (a leveller and a mechanical extensometer – Figure 22) allows obtaining important information about structural behaviour of a historical building, related to environmental conditions and unexpected events influencing its work.

5. Composite strengthening and protection of masonries

5.1. Application of composite materials in engineering practice

Composite strengthening systems are effective tools using for improving of masonry structural strength in civil engineering. Two RILEM Technical Committees (TC 223 MSC and TC 250 CSM) have been working for the last decade on checking if composite strengthening of masonries can help in protection of ordinary and historical masonries.

These systems are quickly developed especially in Italy, where earthquakes demolished many historical city centres (L'Aquila, Mirandola, Amatrice).

The composite strengthening started to be use in retrofitting of real historical structures (Figures 24 and 25).

It should be pointed that effectiveness of these systems was confirmed in laboratory tests but there is low knowledge about durability behaviour of them and long resistance to many earthquakes.



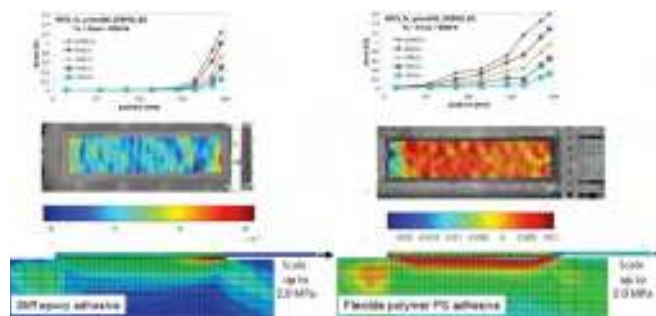
Figures 24. Composite strengthening of an historical building in L'Aquila (Italy), courtesy of Kerakoll.



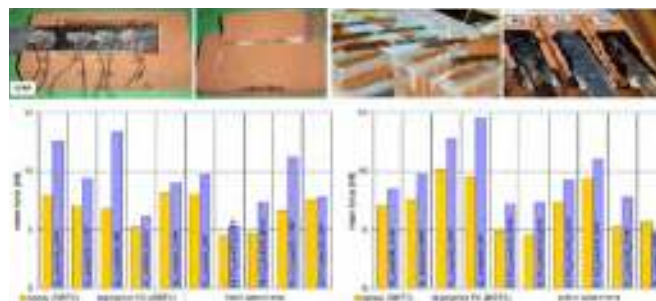
Figures 25. Seismic retrofitting of a damaged masonry building in San Marino, courtesy of Kimia.

5.2. Difference in work of composites bonded using stiff and flexible adhesives

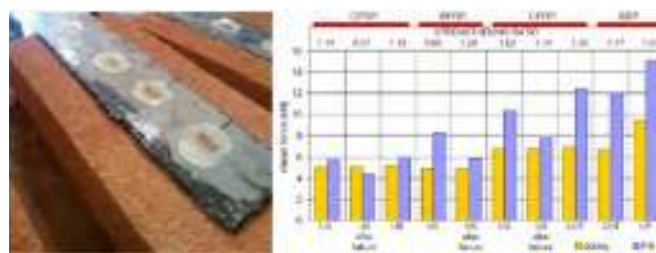
Typically, stiff adhesives of high strength are used (epoxy resin or mineral mortars) in bonding of composite strengthening to masonry substrates (FRP – Fiber Reinforced Polymers), which generates stress concentrations responsible for failure mechanism. The use of more flexible adhesives made of polyurethanes (FRPU – Fiber Reinforced Polyurethanes) allows reducing peaks of stress concentrations by redistributing them along the whole bonding area, what results in higher loads carried by such composite strengthening systems. Flexible adhesives allow collaborating even low strength fibres and low strength brittle substrates, introducing larger strength, ductility and bearing capacity, thus making them safer in exploitation. Digital Image Correlation (DIC) measurement showed and confirmed that the shear load increase in composite strengthening systems with flexible adhesives is caused by more even stress distribution along the bonding length than in stiff and brittle mineral adhesives (Figure 26).



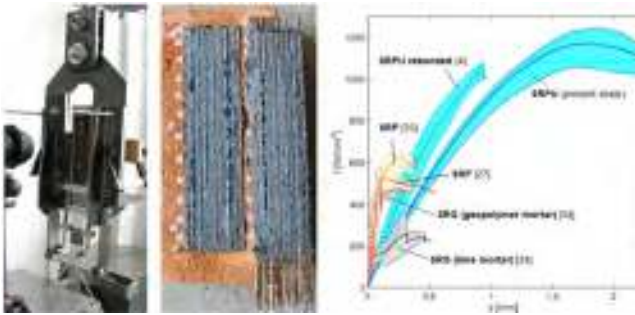
Figures 26. Left, shear stress distribution generated by stiff and right, flexible adhesive bonding composite strengthening to a brick substrate [10].



Figures 27. Comparison of ultimate loads obtained by various laboratories on the repaired brick and prism specimens tested in shear test with stiff epoxy and flexible polyurethane PS adhesives [16].



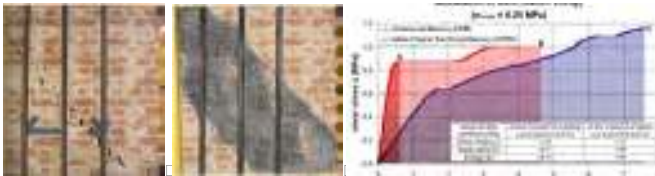
Figures 28. Comparison of ultimate loads obtained by various laboratories on the wet-lay-up brick specimens tested in shear test with stiff epoxy and flexible polyurethane PS adhesives [17].



Figures 29. Comparison of characteristics of steel composite strengthening bonded on polyurethane PS (SRPU), epoxy (SRP) and mineral (SRG) adhesives to brick substrates [18].



Figures 30. Reversibility of polyurethane PS flexible adhesive – easy removable with a spatula [19].



Figures 31. Comparison of obtained static shear test results on the cracked masonry wall strengthened by FRPU with glass grids bonded on polyurethane PS [10].

Composites detached with thin layer of a brick substrate (bonded primarily by stiff epoxy resin) were repaired by bonding again, using flexible polyurethane PS adhesive (Figure 27). In all cases significant increase of ultimate load was observed on brick and prism tested up to failure during international tests carried out by European laboratories [16].

Comparison of wet-lay-up composites-to-brick strengthening systems, carried out on weak and strong Italian brick substrates using stiff epoxy resin and flexible polyurethane PS adhesives, was done also in frame of the international cooperation. The weak brick systems with polyurethane adhesives manifested higher ultimate load values than the same with epoxy adhesives (Figure 28), as in the case of repair. Comparison of SRPU (Steel Reinforced Polyurethanes) with SRP (Steel Reinforced Polymers) and SRG (Steel Reinforced Grouts) is presented in Figure 29.

It is visible that polyurethane PS flexible adhesives (SRPU) assure higher strength capacity and ductility than epoxy (SRP) and mineral (SRG) adhesives used with steel fibers. Moreover, damage of the bond is in the polyurethane adhesive, thus valuable masonry substrate is protected against damage. In addition, the rest of damaged polyurethane adhesive can be easily removed from the masonry substrate using a spatula (Figure 30), thus requirements of reversibility in strengthening of historical structures is fulfilled. Durability aspects of SRPU systems were also investigated with obtaining positive results [18].

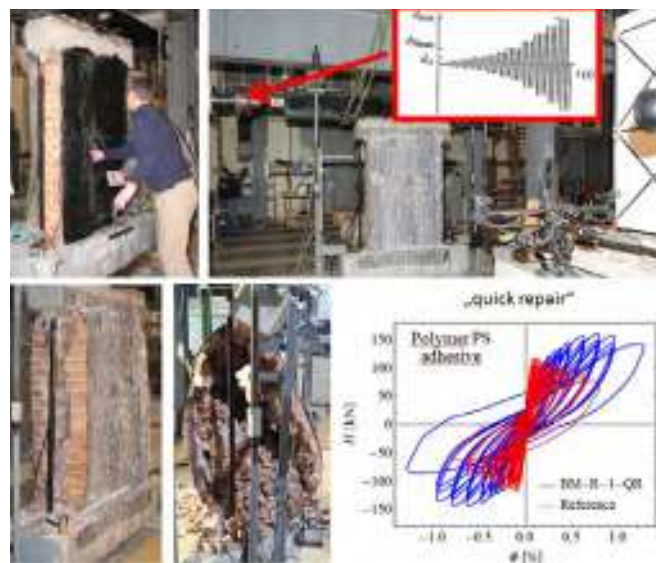
5.2. Laboratory tests of FRPU effectiveness on semi-real scale specimens

Efficiency of highly deformable adhesive polyurethane PS ($E = 16 \text{ MPa}$ and $\epsilon = 40\%$) was also examined during quasi-static laboratory research. A shear test on a cracked masonry wall (pre-compressed vertically) was carried out using composite strengthening made of four layers of glass fibre grids, fixed to the both surfaces of the masonry (after crack appearance) using polyurethane PS (FRPU). The composite mats in

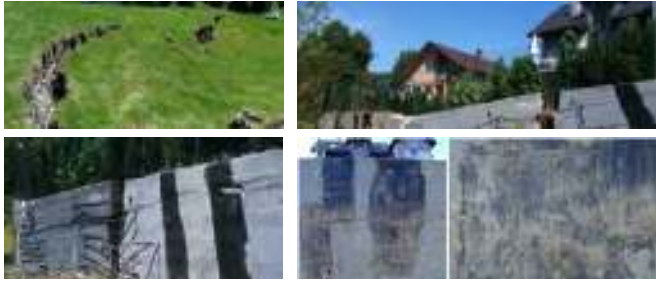
the form of diagonal bands were fixed without filling the crack, so local destruction of masonry in the crack surrounding (caused by the high level of stress concentrations) was expected. The final damage occurred only in the composite in form of glass fibre rupture, because of very good adhesion of flexible polyurethane PS to masonry substrate. Results obtained from the shear tests showed that strength of the repaired masonry specimen exceeds 133% of the original one and increase of the ultimate shear strain was over 12 times and increase of energy dissipation capacity over 16 times (Figure 31).

Efficiency of the FRPU strengthening systems for earthquake was tested in laboratory using cyclic shear tests. At the beginning, a masonry wall was damaged by cyclic shear load and then quickly strengthened with a deformable composite system, which consisted of four layers of glass fiber mesh and polyurethane PS adhesive. It was applied on the cracked surfaces of the wall. In this way, a quick repair of a damaged wall during an earthquake was simulated. The strengthened wall was tested up to collapse under shear load increasing cyclically, 23 hours after application of the composite system. Even at collapse, the bond between the coating and the masonry was not lost. The collapse was due to compressive crushing of the masonry, when the bricks were crushed into smaller pieces, and the connection between the outer layers was severed. After repair, the wall's capacity increased with ratios: resistance capacity by 1.29, rotation capacity by 1.61 and hysteretic energy capacity by 1.73. On the contrary, the global stiffness of the wall after quick repair was reduced by about 24% (Figure 32).

Quick and safe protection of masonry structures just after earthquake, but before aftershock, is a crucial aspect for emergency teams. They require a quick and easy in using strengthening method, which can provide enough strength and ductility properties to the masonry structure weakened by an earthquake. The proposed FRPU composite strengthening system fulfils these requirements of earthquake emergency teams. High efficiency of the FRPU strengthening system consisted of glass mesh and polyurethane PS adhesive was demonstrated during emergency repair of a cracked concrete retaining wall. The soil movement broke this wall, which protected a building founded on an active slope. The applied deformable composite strengthening allowed stopping widely opened and actively increasing cracks in a few hours after fastening.



Figures 32. High efficiency of FRPU repair strengthening made of glass grids bonded on polyurethane PS, examined on masonry wall in a cyclic shear test [10].



Figures 33. The real application in quick emergency process of FRPU system on cracked concrete retaining wall, being damaged and deformed on active slope [10].

Finally, the composite system held back the moving parts of cracked walls, dissipating huge amount of energy – the observed failure mode was in the form of fibre rupture instead of debonding (Figure 33), similar in behaviour to that presented in Figure 31.

6. Conclusions

In this paper innovative intervention methods used in retrofitting and protection of heritage masonry structures against earthquakes were presented. Most of presented ones are based on the innovative Polyurethane Flexible Joint (PFJ) method, which uses special kind of highly deformable polyurethanes for constructing structural joints (PUFJ) able to carry loads and transfer high deformations simultaneously or for bonding of composite strengthening to masonry substrates (FRPU, SRPU). The PFJ (by its deformability and low stiffness) is able to reduce stress concentrations and redistribute them over larger bonding area. This ability allow protecting weak (heritage) masonry substrate against damage and carry higher loads. Advantageous properties of the PFJ systems were confirmed in various laboratory and in situ tests,

which were presented in this paper. Their efficiency was compared with stiff bonding systems, proofing usefulness of the PFJ systems also in real applications on historical structures. Using of innovative retrofitting systems has to be controlled by monitoring of their behavior during exploitation. Monitoring systems should be also used in retrofitting process by group of experts of various specializations. It is useful during condition assessment of historical structure before decision on an intervention, retrofitting process and after the intervention to evaluate effectiveness of it.

The case study presented in this paper indicates that simple monitoring systems can help to understand structural behavior of historical structures in relation to environmental and other external influences.

The PFJ systems have big potential in seismic protection of masonry structures. Flexible joints can be constructed before an earthquake as well as between a main shock and an aftershock. The period of time before an aftershock, during which suitable temporary repairs could be made, is rather very short. If damaged structures are repaired or strengthened quickly, using fast and effective protection methods (PUFJ or FRPU), the life of victims and rescue teams as well as damaged structures would be protected against total or partial collapse. It is believed that it is better to protect than to repair or reconstruct, especially if valuable heritage structures are taken into consideration. Short time before an aftershock does not allow for choosing a final repair strategy and waiting for the authorities' consent about application of strengthening methods (time-consuming in present). In most cases, earthquakes devastate large areas and

many structures, and there is no time to think about the most suitable repair for a particular structure. However, there could be enough time to apply quick protection methods (PUF) or FRPU), even temporarily because they are removable (important especially for historical structures). Despite the flexibility of the polyurethane, the structural joint as well as the composite-to-substrate adhesive bonds would be capable of transferring significant loads during in-plane and out-of-plane excitations.

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Structural Restoration of Daphni Monastery, a World Heritage Monument in Greece

Based on ISCARSAH Approach

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Abstract

The fortified Daphni Monastery, inscribed in the World Heritage List of Unesco since 1990, is one of the major middle- Byzantine monuments. The earthquake on the 7th of September 1999 that affected the region of Attica provoked severe damage to the buildings and the fortification walls of the Monastery, as well as to the ruins of the archaeological site. Extensive damages were as well caused to the Katholikon (main church) and its precious mural mosaics. Thus, large scale emergency measures were applied, to protect the monument from further deterioration and to provide safe working conditions for the Personnel.

Besides, due to the extent of damage, large scale structural restoration and conservation interventions were carried out, not only to the Katholikon and its mural mosaics, but also to all other standing constructions and archaeological ruins. In this seminar, the overall structural restoration project of the Monastery and a concise synopsis of the structural pathology, assessment and interventions in the series of cells of the south courtyard, the fortification walls and the Katholikon will be presented. Special emphasis will be given to the holistic methodology and the “design in process” adopted for the structural restoration of the Katholi-

kon, as this case study constitutes a very good example to highlight the ISCARSAH guidelines.

Keywords: Byzantine monastery, Masonry, Mosaics, Earthquake, Structural restoration

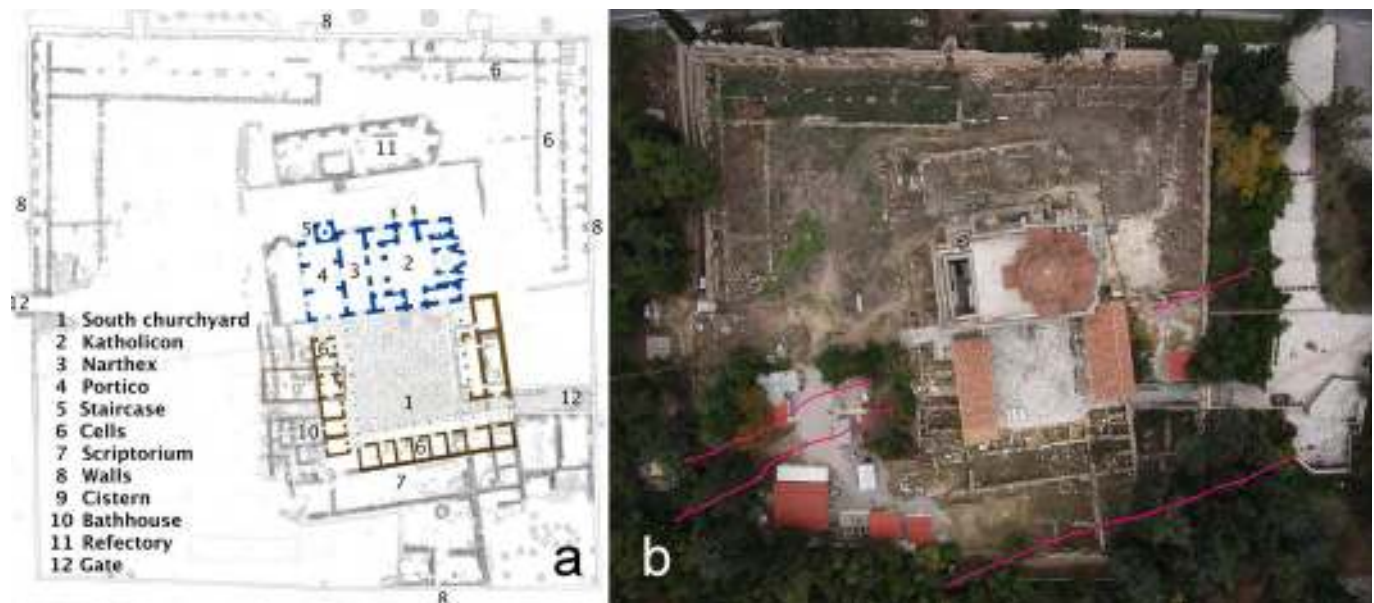
1. Introduction

Daphni Monastery is one of the major middle-Byzantine monuments [1-4], situated in a forestry area at 11 km from Athens on the Sacred Way of the Classic Antiquity, from Athens to Eleusis. (Figure 1). It comprises various buildings laid out in an almost square plan, constructed over a period starting from the 11th century and onwards [3]. Currently, most of the buildings are in ruins, apart from the Katholikon, the two internal ranges of cells, the cistern and the northern fortification walls (Figure 1).

The earthquake on the 7th of September 1999 that affected the region of Attica provoked severe damage to the buildings and the fortification walls of the Monastery, as well as to the ruins of the archaeological site. Immediately after the struck of the earthquake, a multidisciplinary working group was formed by the Hellenic Ministry of Culture (HMC), with the assignment to



Figures 1a. The Monastery from S-W during restoration and 1b. after the earthquake [8] © Hellenic Ministry of Culture.



Figures 2a. Topographic plan of Daphni Monastery, 2b. Soil fractures surveyed after the 1999 Athens earthquake. Masonry collapse has occurred in all areas of their intersection with the perimeter wall [8] © Hellenic Ministry of Culture.

do the necessary inspections, assess the nature and the significance of damage and prepare in collaboration with all competent services of the Ministry a strategic plan for the protection, restoration and conservation of the monument, its mosaics included (composed by N. Delinikolas, Architect Engineer, A. Miltiadou-Fezans Civil Engineer, E. Tsofopoulou Archaeologist, D. Chrissopoulos Conservator and N. Minos Conservator). Moreover, a multidisciplinary Scientific Committee was also nominated to assist the Services of the Ministry to this purpose (composed by Professors: Ch. Bouras, T. P. Tassios, E. Mariolakos and N. Zias).

The first inspections made evident the need for large-scale emergency measures to protect the monument from further deterioration and to provide safe working conditions for the Personnel [5]. Besides, the extent of damage indicated that large-scale structural restoration and conservation interventions should be carried out, to reach the consolidation of all standing constructions.

In this paper, the overall structural restoration project of the monastery and a concise synopsis of the structural pathology, assessment and interventions in the cells of the south courtyard and the fortification walls will be first presented. The case of the Katholikon will be discussed in a more detailed way, as it constitutes a very good example to highlight the ISCARSAH guidelines [6]. Special emphasis will be given to the “design in process” adopted for the structural restoration of this World Heritage monument, as this approach constitutes a key issue for the rational design and application of the optimum interventions [4, 7, 8, 9]. Given the importance of the monument and the complexity

of problems to be faced, this approach led to the partition of the restoration works in two phases, in combination with the installation of an earthquake monitoring system [10]. Thus, it was possible to inspect and document hidden areas of the structure [8, 9], to experimentally investigate the behaviour of masonry before and after grouting [11] and to record the dynamic characteristics of the structure, before, during and after the implementation of the first phase interventions, thereby assessing their impact on its overall behavior [10]. The aim of this endeavour was to design in a rational way the optimal interventions, considering all the values of the monument, and above all the protection of its mosaics [8, 9]. To this end additional investigations were necessary to improve the knowledge level, in order to perform all necessary numerical analyses and safety checks, with calibrated models and reliable data, avoiding conservative assumptions (both for actions and resistances). Such assumptions would lead to extensive interventions, that might not be needed and that would inevitably alter the architectural value of the monument.

The first phase of works comprises all interventions considered necessary to achieve the best possible repair and strengthening of masonry elements (stitching of cracks, deep re-pointing where necessary, systematic hydraulic lime grouting, local reconstructions, etc.) [4, 7]. The second phase comprises strengthening interventions, designed and selected as optimal, to improve the overall behaviour of the entire structure, such as: ties and struts, “diaphragmatic” structures over the extrados of the groin-vaults and in the exonarthex, bracing of the windows of the drum of the dome, confinement of structural elements, etc. The

second phase consisted of interventions aiming at reducing the vulnerability of the structure and the extent of the damage under future seismic events, especially in the upper level, where the mural mosaics are located. A synthesis of the results of all the detailed documentation, in situ and laboratory investigations, monitoring, numerical analysis and safety checks, together with in situ feasibility checks, led to the selection of the optimal interventions, namely those that achieve the required structural performance with the minimum possible impact on heritage values while ensuring the protection of the mural mosaics [12, 13]. In this paper a concise synopsis of the whole procedure followed will be given. The importance of training and that of multidisciplinary collaboration in every day practice, will be also underlined, as they are key issues for the successful implementation of such challenging interventions [8].

2. Presentation of the monastery-main restoration projects

2.1. Brief presentation of the Monastery

In Figure 2a the various buildings of the Monastery are indicated. The surrounding fortification wall, together with a series of cells situated close and in parallel to it, constitute the external yard of the monastic complex. The Katholikon (main church) is dedicated to the Dormition of Virgin and it is decorated with precious mural mosaics of excellent art. The big refectory is situated to the north of the Katholikon, while to the south an internal churchyard is formed by various constructions (ranges of cells, colonnades and outbuildings), that have been renovated or reconstructed several times during

the thousand years of the life of the Monastery. The underground cistern, still in function, and the bath-house are situated at SW of the Katholikon.

2.2. The Monastery is built in a seismic area

The Monastery is situated in a neogene tectonic graben between the mountains Egaleo and Korydallos at the west side of Athens, 150m away from the E-W trending marginal fault between the alpine Mesozoic limestone and the post-alpine deposits. The base of the monument consists of post alpine formations, that is, recent continental deposits of small thickness and lacustrine marls of the Upper Miocene, which form almost flat morphology at the site [14]. As reported by Mariolakos et al. (2000) [14], there is a large-scale slip fracture (trichoid short seismic fractures) developing outside the surrounding wall of Daphni Monastery, along several tenths of meters (see also Figure 2b). The authors underline that all damages occurred during the 1999 earthquake are connected with seismic fractures on the ground, or the extensions of those fractures and are due to the presence of the marginal fault of the graben, along which high seismic energy is released. During the last two centuries, several earthquakes, having a magnitude higher than 6 on the Richter scale, have affected the monument [4, 8, 9], proving that the Monastery is located in a tectonically active area. As a result, partial collapses and extended damage were provoked to all the constructions and the mural mosaics, throughout the centuries. Thus, when the 1999 Athens earthquake occurred (magnitude 5.9R), most of the buildings were in ruins, except of the Katholikon, part of the cells of the south courtyard, the cistern and the northern fortification

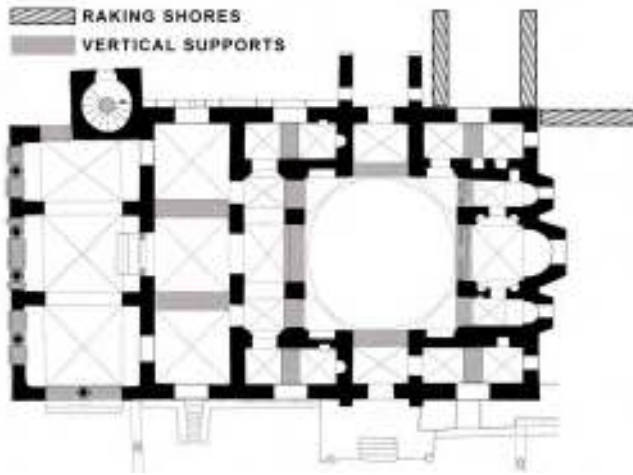


Figure 3. Presentation of emergency measures [5].

wall (Figure 1, 2).

2.3. Emergency measures, surveys, investigations and main restoration projects

To avoid further damage due to the sequence of after-shocks, the following measures were urgently taken: a) special nets were installed at various levels in the interior of the Katholikon near the mosaics to collect the mosaics' tesserae, which could fall down, b) tell-tales were installed across all the accessible severe cracks, c) a crane was used to inspect the upper part of the Katholikon and especially the area of the dome and the drum, as well as that of the exonarthex, d) crumbling and dislocated masonry parts of dilapidated walls of cells were removed to the ground or demolished in a controlled way. Subsequently, on the basis of the first damage survey and the identification of the activated collapse mechanisms of various buildings and ruins, adequate temporary shoring and propping measures were implemented [5].



Figure 4. Raking shores installed in the NE corner [5] © Hellenic Ministry of Culture.

The emergency interventions were designed taking into account specific demands deriving from the importance of the monument and the necessity for implementation of final restoration works without removing the supports and scaffoldings.

Thus, they had to be reversible, easily assembled and allowing for gradual disassembling in the interior, as well as adjustable to the deformed geometry of damaged elements. Moreover, any contact with the vulnerable mural mosaics was to be avoided.

To this end various alternative solutions were examined [5]. Figure 3 shows schematically the retained one. In the NE corner of the building, three double-framed steel raking shores was constructed, as in this area a pronounced tilting of the external walls has been noticed and the telltales installed just after the earthquake, indicated further opening of cracks and a tendency of the corner to detach (Figure 4). In the interior (Figure 5a), and in the exonarthex (Figure 5b), vertical steel props



Figures 5a. Vertical steel props in the interior and 5b. in the exonarthex [5] © Hellenic Ministry of Culture.

were built beneath the main arches, in order to provide vertical support to their cracked structure.

Both the raking shores and the vertical props were self-supported elements, which would be activated only in case of further increase of the deformations of the structure. Thus, between the metal framework and masonry walls, a 12 cm full layer of wooden beams and wedges (together with a 3 mm soft packing) were inserted, to provide good contact with the masonry without harming the surfaces, while allowing relative movement, unless the structure did start to move towards them. Moreover, the upper part of the exonarthex walls and the NW and SW piers were confined using steel plates and bars.

Regarding the drum of the dome, specially designed steel elements were constructed to brace the windows and confine in two levels the masonry piers (Figure 6), taking special care to assure the in-situ assemblage of all these structures without harming the mural mosaics. Furthermore, the following measures have been undertaken: (a) the installation of adequate types of scaffoldings in the interior and exterior of the church, in order to offer safe working conditions for the Personnel, and (b) the removal of the tiles of the roof and the applica-



Figure 6. Dome's windows bracing [5] © Hellenic Ministry of Culture.

tion of temporary water isolation membranes just below them, to protect the cracked extrados of the vaulted structures, and hence the mural mosaics from leaking water.

In parallel to the design and application of the adequate urgent measures, a first series of research programs and investigations were undertaken to record the state of the Katholikon immediately after the earthquake. These programs were aiming at:

- (i) An accurate geometrical and architectural survey of the church, using novel survey methods and digital photogrammetry, in collaboration with the School of Rural and Surveying Engineering of the National Technical University of Athens (NTUA) [15, 16],
- (ii) A preliminary survey of damages, assessment of mechanical properties of construction materials, monitoring of the width evolution of main cracks, and preparation of a first detailed numerical model to be used for preliminary structural analyses, in collaboration with the Laboratory of Reinforced Concrete of

N.T.U.A. [17, 18],

(iii) An investigation and assessment of the physico-chemical characteristics of the construction and pointing mortars, in collaboration with the Laboratory of Construction Materials of the Aristotle University of Thessaloniki [19], and

(iv) A geotechnical investigation of foundation soil [20] and a geophysical investigation of foundation, in collaboration with the Department of Geology of the University of Patras [21, 22].

For the overall consolidation and restoration of the Monastery, a master plan was prepared. One important target was to start, as soon as possible, the works in some parts of the monument, while completing the design of the interventions for other parts. Given the importance of the monument and the complexity of influential parameters, that had to be taken into account, the elaboration of the restoration design of various parts of the monument was assigned to the aforementioned Daphni Monastery multidisciplinary working group, established after the earthquake, composed by representatives of all co-responsible Directorates of the HMC, namely the 1st Ephorate of Byzantine Antiquities (1st EBA), the Directorate for the Restoration of Byzantine and Post-byzantine Monuments (DRBPM), and the Directorate for the Conservation of Ancient and Modern Monuments (DCAMM). That working team was enhanced, by permanent scientific and technical Personnel of the Directorates or temporary Personnel employed by them specifically for each project. In all cases, a close collaboration with the Scientific Committee and the Universities, involved in relevant research, was also ensured. Furthermore, the members of the multidisciplinary working group con-

tributed also to the supervision of the application of the works. Thus, continuity was ensured at all steps of the restoration works of the Monastery.

Moreover, the continuous support of the aforementioned Directorates by the Directorate for Technical Research on Restoration (DTRR) of the HMC was judged as absolutely necessary for the realization of all adequate research, investigations and pilot applications, together with the training of the scientific and technical Personnel in the implementation of novel materials and techniques. On the basis of the aforementioned framework, large scale structural restoration interventions were carried out, comprising the following:

- a) Restoration of the ruins and buildings of the south courtyard of the church and the ruins of the external yard (2003 - 2006).
 - b) Restoration of the Katholikon and conservation of its mosaics (1st Phase 2004-2007, 2nd Phase 2011-2016).
 - c) Restoration of the southern, western and eastern fortification walls and first phase of interventions for the protection of the northern wall (2011-2014).
- Although, in the northern part of the fortification wall, mainly emergency structural measures were carried out until 2016, the restoration study had already been completed and approved. It is hoped that the necessary funding will become available in the near future to allow for the completion of all restoration works of this important part of the monument as well.

3. Restoration interventions of the south courtyard buildings

3.1. General aspects

The 1999 earthquake provoked severe damage and



Figure 7a. West façade of the Eastern range of cells and 7b. emergency measures to the collapsed south wall of first floor [8]
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extended collapses in the constructions of the south courtyard and the surrounding ruins. The buildings at the eastern and western side of the yard are the only ones, among the series of cells, that have survived. However, even those buildings preserve at a very limited extent their original byzantine or post-byzantine form, as they underwent large scale reconstruction in the 19th and 20th centuries (Figure 1b).

Nevertheless, they are very instructive for the architectural history of the monument.

Moreover, they constitute the only buildings in which some indispensable for an archaeological site functions may be hosted (guard's office, reception of visitors, educational programs, exhibition of architectural members, earthquake monitoring control room, etc.). Thus, detailed historical, architectural and structural surveys and analyses started immediately after the implementation of emergency measures and the restoration design was elaborated in 2001 [23]. On the basis of the approved design, the decision was made to maintain the morphology of the internal yard, as it was established in the 1930's, when the long room to host



Figure 8. Main construction phases and interventions of the buildings of the south churchyard [8].

the exhibition of architectural members was constructed. The respective works were initiated in 2004 and were completed in 2006. A detailed presentation of the restoration design and works in this part of the monument being beyond the scope of this paper, the main structural restoration interventions are briefly mentioned herein.

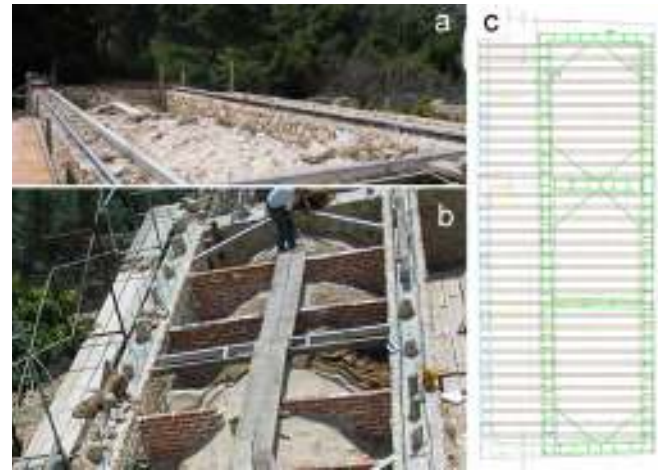
3.2. Structural restoration of the Eastern range of cells

The ground floor of the two-storey eastern range of cells is composed of two vaulted rooms of middle byzantine and post byzantine period, and a third one reconstructed in 1950-1960, after the demolition at the end of 19th century of the entire two-storey southern part of the building, due to heavy damage occurred through the centuries. Only a part of the post byzantine vaulted portico survived at east; the rest of the portico, covered with a wooden roof, was also reconstructed in 1950-1960 (Figure 7a, 8). In the first floor, only two rooms, constructed during the late 19th c., have survived, with many alterations, and repairs and

extremely heavy damage. Both the north and south external walls were partly collapsed during the 1999 earthquake and multiple severe cracks were surveyed in the deformed walls (Figure 7).

Based on all surveys, thorough in situ observations, documentation, structural analysis and safety checks, the following structural interventions were designed and carried out. All masonry walls and the vaulted roof of the ground floor were carefully repaired and strengthened by deep re-pointing, installation of stone or stainless-steel stitching, systematic application of hydraulic grouts, and repair or replacement of timber reinforcements and lintels. Moreover, stainless steel ties and struts were installed in the springing of the existing vaults of the ground floor rooms and anchored on the external face of the west and east walls.

The remains of the late 19th century walls at the first floor were carefully demolished, thus permitting to proceed in a safer way to the reconstruction of the entire first floor structure, including the south part already demolished in the 19th c. Due to the extended damage of the old barrel vaults of the two eastern rooms of the ground floor, it was considered necessary to insert a new floor ensuring diaphragm action, over the extrados of the vaults. The new floor consists of horizontal perimeter beams (made of stainless steel) embedded to the vertical walls at their top, whereas wooden beams, plywood panels and planks are used to provide in plane stiffness (Figures 9a, c). Thus, the seismic behavior of the building was improved, while the vertical loads of the floor were directly transferred to the vertical walls, alleviating the old vaults. As shown in Figure 9b, care has been taken to install vertical stone stitches and dowels, in-between the edges of the wooden beams, in order to



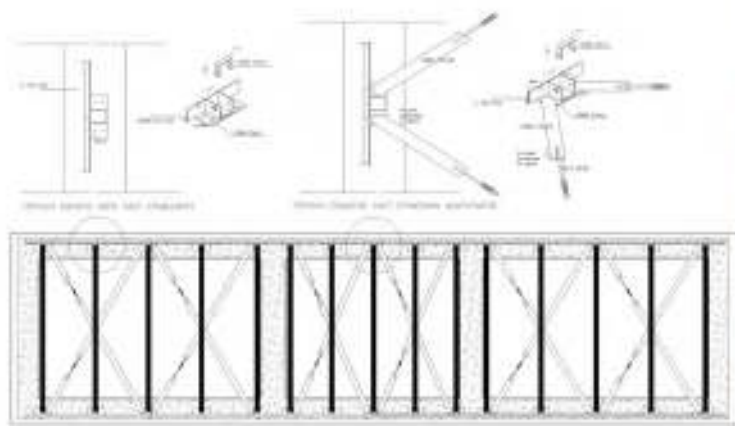
Figures 9. New floor with diaphragmatic function of the Eastern range of cells: 9a. View of the stainless-steel perimeter beams, 9b. View of the vertical stone stitches and stainless-steel dowels to connect old and new masonry, and 9c. Drawing with the lay out of the stainless-steel perimeter beams and the wooden beams [8].

better connect the lower old part of the stone masonry walls with the upper new one.

Furthermore, a wooden roof of sufficient in-plane stiffness was constructed (Figure 10a) to cover the first floor of the building and improve its seismic behaviour. In Figure 10b the Eastern range of cells after restoration is shown.

3.3. Structural restoration of the Western range of cells

The Western range of cells of the south courtyard is a single-storey building, comprising, in its south end, two rooms covered with barrel vaults, dating back to Post-byzantine period, and one long room, covered with a timber roof and internally partitioned by (lower than the full height of the building) transverse walls (Figure 1, 2). That room, built in the 1930's, to accommodate



Figures 10a. Drawings of the diaphragm installed at the roof of the Eastern range of cells, 10b. Southwestern façade of the Eastern range of cells restored [8] © Hellenic Ministry of Culture.



Figures 11. View of the Western range of cells, during the consolidation works of old masonries and of archaeological remains found underneath the 1930's construction (left) and restored (right) [8] © Hellenic Ministry of Culture.

the exhibition of sculptured architectural members found in the Monastery, was of poor construction quality. Furthermore, it was founded on the Cistern, as well as on other archaeological remains (byzantine

bathhouse), without previous repair thereof. To those two factors is attributed the occurrence of extended damage during the 1999 earthquake. It was therefore judged absolutely necessary to demolish the 1930's

walls, carry out the necessary archaeological research (Figure 11 left) and then repair and strengthen in the best possible way, all the archaeological remains, before reconstructing this range of cells of the churchyard. Care was taken to provide adequate connection between the preserved and the reconstructed portions of the building, while openings were not provided to the west façade, thus offering more space to cover the needs of the exhibition of archaeological findings. The construction of a timber roof (ensuring diaphragm action) and the installation of ties-struts in the vaulted rooms in the southern part complete the scheme of interventions (Figure 11).

4. Restoration interventions to the fortification walls

The thorough historical, architectural and structural documentation of the external yard [24], has revealed that the fortification walls were constructed following an overall original design, which was quite austere and has respected the rules of defensive architecture (Figure 12).

Nevertheless, the poor quality of large stone blocks stones (probably, in second use) used for the construction of the lower part of the walls, along with their rather limited thickness (1.10 m for the wall alone and 1.60 m together with the piers of the arcades), compared with their length (95.80 m X 97.50 m) and their height (9.10m for the walls and 9.7m for the towers), may interpret the vulnerability of the fortification walls and, hence, their historical and current pathology. The study of their historical pathology allows for the assumption to be made that, due to the high seismicity of the area, the fortification walls have suffered significant

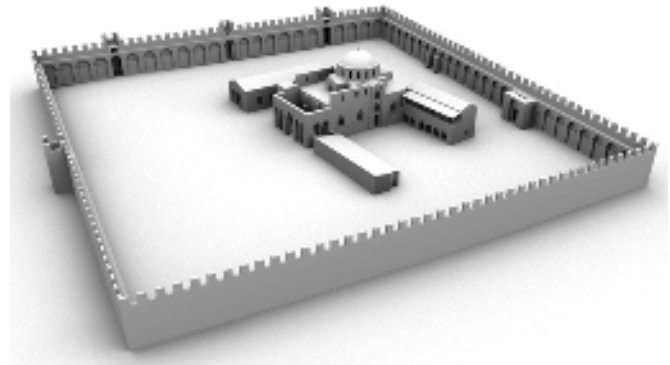


Figure 12. 3D representation of the fortification wall, the Katholikon, the refectory and the cells of south courtyard [8].

damage, as early as in the 13th century. Most probably, they have never been systematically repaired and restored, as there is no evidence of more recent construction phases; only locally some post-byzantine interventions were identified.

From that period to our era the fortification was repeatedly damaged by earthquakes, without essential works to reinstate the collapsed parts and restore its continuity and integrity. Due to the accumulation of damage, only the northern part (perhaps, thanks to the beneficial effect of towers, see Figure 12) preserves the original height and morphology, although cracked, deformed, locally collapsed and altered due to interventions applied mostly during the last century. In fact, during that period, in the eastern, southern and, to a smaller extent, in the western part, rubble stone masonry was built over the ruins of the byzantine walls to provide to the walls the height that was necessary for the protection of the monastery. This was necessary, especially at the northern part of the fortification, as the level of the road parallel to the wall was set higher than before. The use of cement mortars,

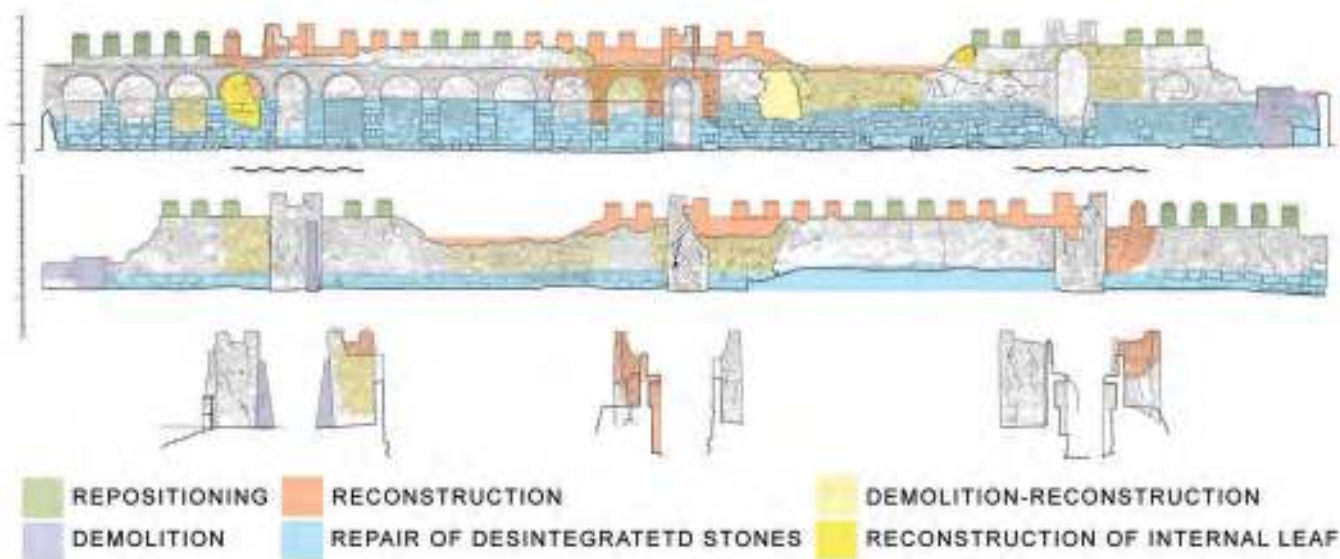


Figure 13. Masonry interventions to the internal and external façade of the Northern wall [8].

both for re-pointing and filling missing parts of the stones, proved to be rather unfortunate, as it led to the disintegration of stones and to further deterioration of the fortifications.

The 1999 earthquake provoked severe damage and local collapse of several parts of the ruined walls. According to the design approved in 2010 [24], rubble stone masonry portions constructed during last century were demolished, whereas repair and strengthening of old masonries was systematically carried out, using lime-pozzolan or hydraulic lime mortars and hydraulic lime grouts.

For the northern part, the design prescribes a series of interventions to masonry, aiming at the preservation of the old remains (Figure 13). Moreover, a stainless-steel tie-rod was installed in the arcade along with stainless steel elements to confine, support or connect the var-

ious parts among them. Due to the crumbling situation of portions of the northern wall, emergency measures had to be first implemented, in order to prevent further collapse and provide safe working conditions to the Personnel.

5. Structural restoration of the Katholikon

5.1. Brief presentation of the Katholikon of Daphni Monastery

The Katholikon belongs to the octagonal type and preserves large part of the original mural mosaics of excellent art. It comprises the main church, the sanctuary, the narthex and four chapels, which complete its orthogonal plan. In the western part, only the perimeter walls of an exonarthex or portico and those of a spiral stairway tower leading to the upper floor have survived (Figure



Figure 14. Left, N-E view of the church and right, view of the dome, after the recent restoration works © Hellenic Ministry of Culture.

14). The central part of the main church is cross-shaped in plan, the hemispherical dome rising over its square core. The dome has 7.9 m internal diameter and is 16.3 m high, and rests on an almost cylindrical drum with 16 piers and 16 vaulted windows (Figure 14). The dome and its drum are carried by eight pendentives and eight arches (four semicircular and four embodied in the squinches of the corners), forming an octagon and achieving in this way the transition from circle to square. Thus, twelve piers (laying out in a square plan), provide support to the dome together with the groin vaulted arms of the cross, situated in a higher level (Figure 14 left). All the other parts of the monument are covered with byzantine groin vaults.

The exterior faces of the vertical perimeter walls are built according to the enclosed brick system (stones with bricks around them). Two different masonry construction types are followed, corresponding to their lower and upper parts [4, 17]. Both types of the perimeter walls are constructed according to the three-

leaf masonry type, with varying widths of their leaves [25]. Plain brick masonry was used for the construction of all the windows and doors (Figure 14 left), as well as of the entire vaulted roof and all arches, apart from those of the west façade of the portico, where curved stones were used by the Cistercians Monks. In order to proceed to the architectural and structural investigation and analysis and the design of interventions, a thorough geometrical and architectural survey of the monument at a general scale of 1:25 was decided. Furthermore, the recording of its mural mosaics at a very large scale (1:5) was considered necessary, in order to serve the need for a very detailed documentation and design of conservation interventions of the mosaics by the Conservators [9].

Besides, the requirements included, among others, the raster development of those mosaics, which are constructed on developable, i.e. cylindrical or conical, surfaces. The aforementioned research program was carried out with the close collaboration of experienced



Figure 15. W and S façade: 1st construction phase. 3D representation [4].

scientific Personnel of the DRBPM and 1st EBA of the HMC [15, 16].

5.2. Construction phases-Pathology- Qualitative Interpretation-Preliminary Analyses

5.2.1. Construction phases and historical pathology

Located in a tectonically active area, many intensive earthquakes damaged Dafni Monastery throughout the centuries (from 11th c. to our era). Partial collapses and extended damage were provoked to the structure and the mural mosaics, which led to major interventions. Detailed presentation of the construction phases and the long history of the monument, including the extensive interventions undertaken (mainly during 13th -15th c. and the end of 19th and during the 20th c.) are reported in [4, 7, 18].

Two main phases of construction have been identified in the Katholikon of Dafni Monastery, both dating back to the middle Byzantine period. Although the construc-

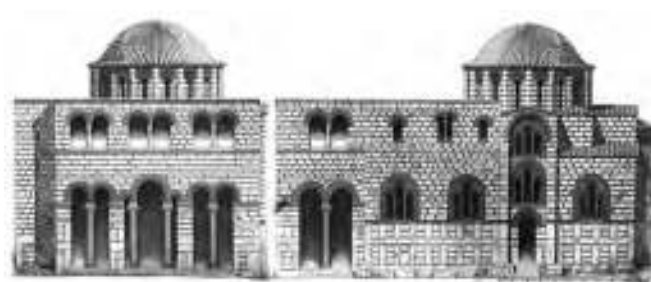


Figure 16. W and S façade: 2nd construction phase. Graphical representation of Stikas modified by Delinikolas [2, 4].

tion type of masonry is similar in the two phases, the composition and final texture of pointing mortar used is different, thus allowing for the two phases to be identified in situ. The first phase comprises the main church and the narthex (Figure 15). A reddish mortar containing crashed bricks was used for the pointing of the joints of the monument's external façades. At a second phase, a portico or exonarthex was added to the church (in its west part); masonry piers in the corners and marble ionic columns in the middle were used to support the arches of the added perimeter walls. Over the portico and the west part of the church a first floor was also added (Figure 16). Finally, a spiral staircase tower was constructed in the NW corner of the narthex. A whitish mortar with a yellowish patina was used for the pointing of the joints of the external façades.

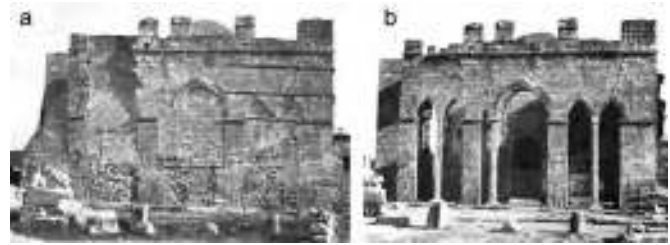
Severe damage was provoked to the church during the 13th century, when Athens was occupied by the Franks. The first floor and the three groined vaults of the exonarthex have probably collapsed, provoking severe damage to the rest of the structure. The narthex, the dome and its supporting system were as well severely damaged. Thus, extensive interventions were undertaken by



Figure 17. Graphical representation of the W façade of the portico as restored by the Cistercians [4].

the Cistercian monks mainly in the west part of the monument (13th-15th c.). The upper part of the west wall and the majority of arches, were reconstructed with curved stones following the gothic style (Figure 17). Furthermore, the exonarthex was covered with a timber roof and bastions were built on the top of the walls. The remains of the constructions of the first floor, the narthex and the rest of the church were also locally repaired. These extensive interventions in the west part of the Katholikon can be considered as the third construction phase of the building.

During Ottoman occupation strong earthquakes provoked further deterioration of the damages of the entire church. Cracks appeared on the vaulted roof of the crypt and the narthex, whereas severe damage and partial collapse occurred to the exonarthex and the spiral staircase. Thus, the arches of the exonarthex perimeter walls were filled with masonry, and extensive alterations took place in the west part of the monument (Figure 18a). During that period some of the ionic columns were taken by Lord Elgin to England.



Figures 18a. West façade of the exonarthex before and 18b. after Stikas restoration interventions [2] © Hellenic Ministry of Culture.

The structural condition of the monument was further deteriorated during the 19th and the 20th centuries, due to numerous earthquakes occurred and major interventions were applied for the preservation of the monument. One should mention the 1889 and 1894 earthquakes (estimated magnitudes 6.7 and 7.0 on the Richter scale), the 1914 earthquake (M 6.0R), the 1981 earthquake (M 6.7R) and finally the 1999 one (M 5.9R). After the 1889 earthquake, the heavily damaged dome and its drum were removed and reconstructed. Three concentric iron rings (I-beams) were inserted at the base of the drum of the dome, as identified during the recent restoration works. Before removing the old dome, the mosaics were detached and, after their conservation, partially re-attached to the monument by Italian Conservators. During these interventions, the lime-based mortar of the mosaic's substrata was replaced with a hydraulic lime based one. The 1894 strong earthquake caused further damage; even the recently reconstructed dome and its drum were once again cracked. Extensive interventions took place to the west part of the monument, including the reconstructions of almost the entire narthex [26]. In the period of 1897 to 1907, two stone buttresses both

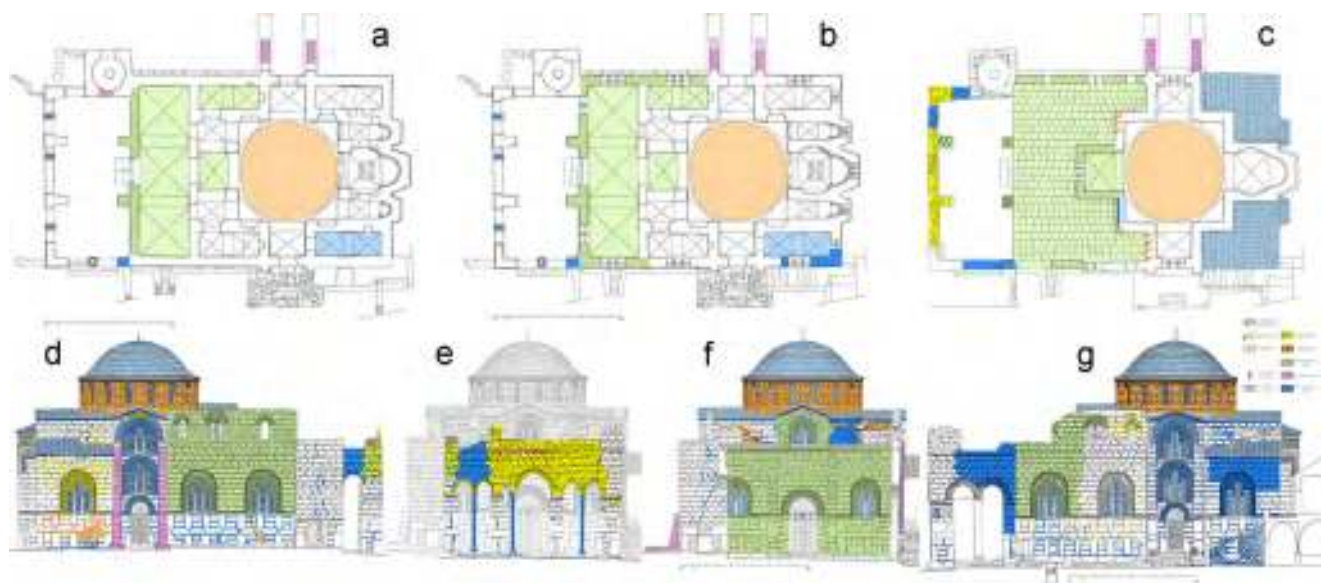


Figure 19. Historical pathology and past interventions: plans at the level of a) entrance doors, (b) lower windows and c) squinches, d) N façade of the church, e) W façade of the exonarthex, f) W and g) S façade of the church. All uncoloured joints still preserve the old Byzantine pointing mortars [adapted from 9].

sides of the north entrance to the church were constructed, while metallic trusses (fixed on new foundation) were installed both sides of the south-central piers, together with iron plates and bars inserted at three levels, in order to confine them. In 1914, the suggestion is made for external confinement of the central cupola with an iron ring. This solution, however, was applied 40 years later.

In that period of 1955-1960 the ottoman interventions were removed from the exonarthex (Figure 18b) and the Cistercian construction phase was restored by Stikas [2]. The collapsed part was rebuilt and the missing ionic columns were replaced by masonry piers. A concrete beam was installed in the interior of the reconstructed parts, without affecting the byzantine remains of the SW pier.

The upper part of the SE chapel was also reconstructed, while extensive local re-pointing took place in various areas of the Katholikon, using mainly cement based mortars. Fortunately, the non-deteriorated old mortars were not removed. Thus, they survived to our days in better condition than those used by Stikas, which presented extended decay and during the recent restoration works were removed. In the 1960's an external upper metal tie-rod, confining the hemispherical dome near its springing level was also installed by Solomonides [4]. In Figures 19, the historical pathology and past interventions are presented on the digital survey drawings of the monument. The reconstructed parts are shown with solid colours. Yellow: 13th-15th c., orange: 1891, green: 1895, purple: 1897-1907, dark blue: 1955-60. The 1981 earthquake (Alkyonides islands) caused

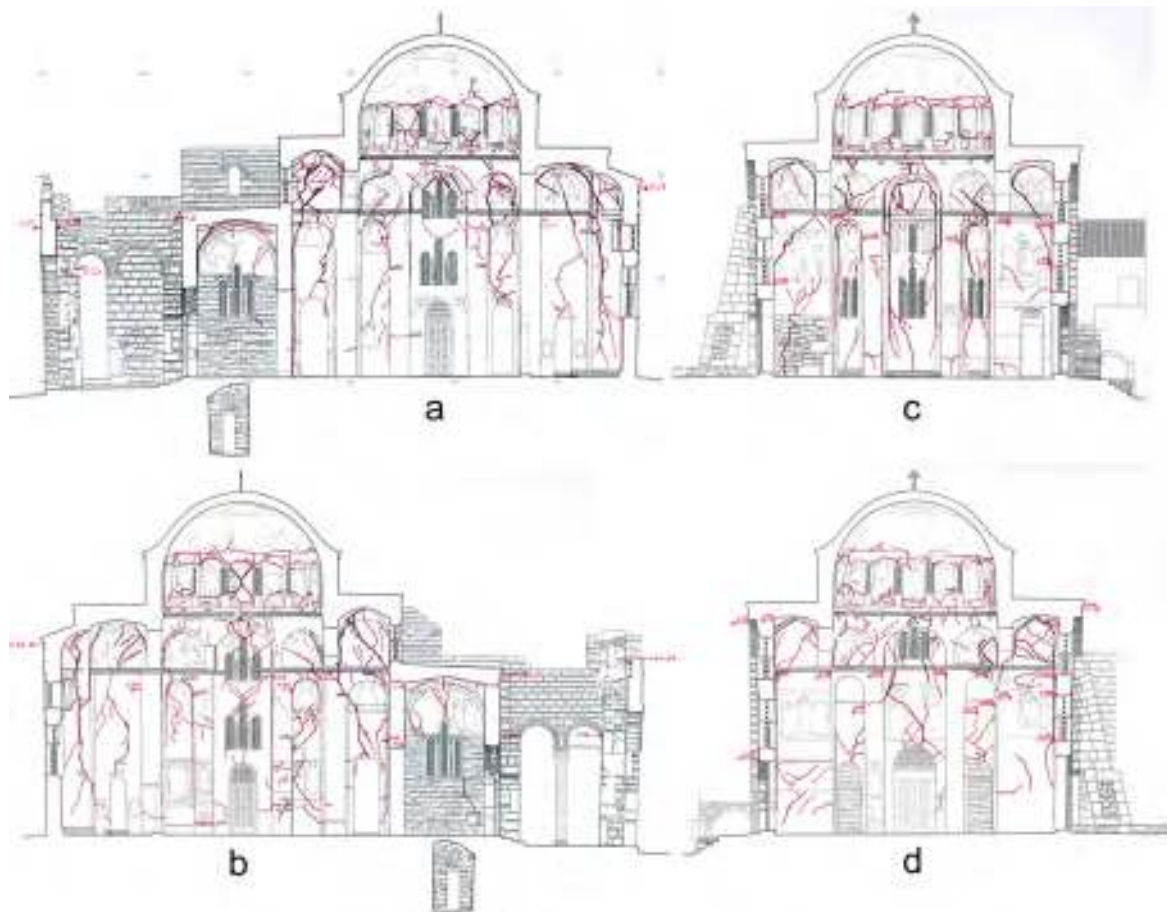


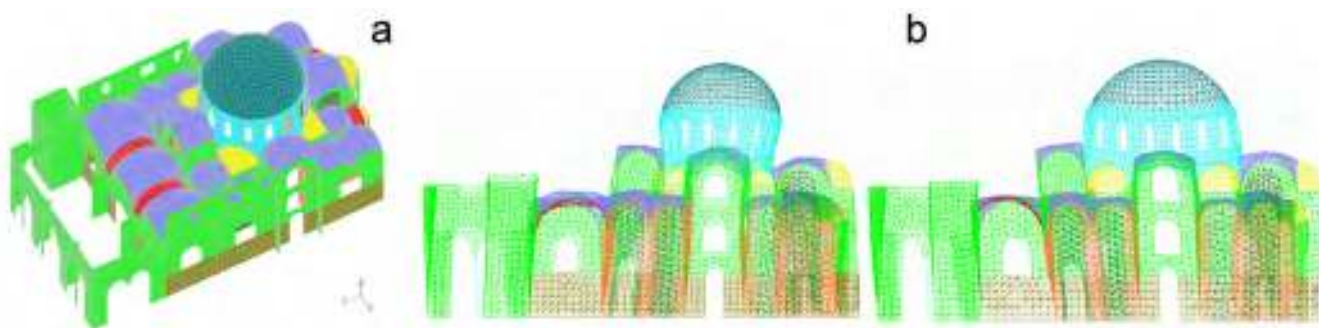
Figure 20. Pathology after 1999's earthquake presented on central sections of the structure: a) East-West section, view to North, b) East-West section, view to south, c) North-South section, view to East and d) North-South section, view to West [adapted from 9].

numerous hair cracks to the building and severe damages to the mosaics. Thus, the pathology observed in the monument after the 1999 earthquake was only partly due to this severe earthquake.

5.2.2. Pathology after the 1999's earthquake, qualitative interpretation and preliminary analyses

After the 1999 Athens earthquake, an extensive net-

work of shear and bending cracks appeared on the walls and piers of the monument, whereas numerous old cracks (due to previous earthquakes) increased in length and width (Figure 20). Severe structural dislocation and outwards movement of the walls was recorded in the NE corner of the main church ($\sim 14\text{cm}$ to the N and $\sim 10\text{cm}$ to the E). Significant out-of-plane displacement of the N and S arms of the cross ($\sim 16\text{cm}$



Figures 21a. The numerical model prepared for parameter analyses, 21 b. Deformed shape of the structure looking from the S (dynamic analysis, seismic action along the longitudinal axis) adapted from [9].

and 21 cm respectively), and of the free-standing west wall of the exonarthex were also recorded, due to further deterioration of previous deformations (~16 cm in the corners and ~25 cm in the middle). The damages were more extensive in the higher parts of the structure, especially in the sanctuary, the arms of the cross and all the arches below the dome area.

As shown in Figure 20, the NE and NW small arches just below the squinches presented severe dislocation near their crown, followed by out of plane deformations of the squinches themselves. The structural condition of the dome (reconstructed at 1891 and damaged soon after its reconstruction at 1894) was assessed as extremely critical immediately after the earthquake. Horizontal cracks have appeared along the perimeter of the drum (both at its base and top). In the piers of the drum that are situated perpendicular to the East-West direction, horizontal cracks (due to out-of-plane bending) have opened at their top and bottom. In the piers that are situated parallel to the E-W axis, diagonal or bi-diagonal (shear) cracks have appeared. In the intermediate piers, mixed type of (less severe) cracks were observed.

This pathological image does seem to confirm seismological data regarding the predominant direction of the 1999 earthquake. Thanks to the external upper metal tie-rod, confining the hemispherical dome near its springing level, the occurrence of severe cracking of the dome's shell was prevented.

One of the major steps of the assessment of an existing structure is the analytical reproduction of its pathological image. In the case of the Katholikon, preliminary linear parameter analyses were performed, as a means for selecting adequate emergency interventions and as a first attempt for the numerical verification of the pathological image of the monument. Thus, for the preliminary analytical study, using the computer code ACORD, the structure was modelled by shell elements (Figure 21a), whereas the mechanical properties of elements belonging to various parts of the structures were assumed on the basis of the available data for the construction materials. Linear elastic analyses were performed for various combinations of actions (self-weight alone or combined with seismic action). Both static and dynamic analyses were performed. These analyses permitted to clearly distinguish the

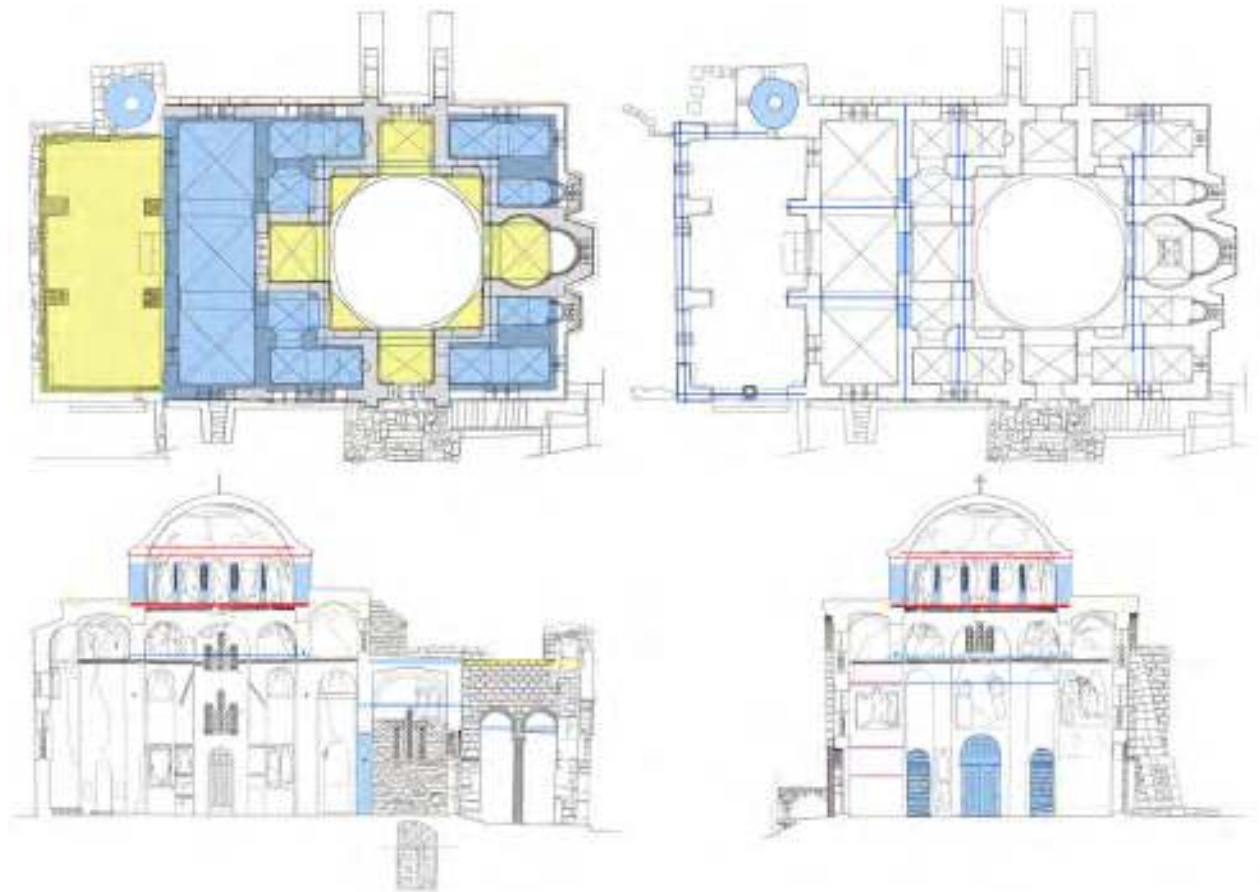


Figure 22. Strengthening interventions to improve the overall behaviour of the structure, approved in principle in 2003 for further elaboration. (blue and yellow surfaces: areas to install “diaphragmatic” structures, red and blue lines: areas for ties-struts installation or circumferential confinement) [8].

vulnerable areas of the structure and the obtained results were conform with the qualitative interpretation of the pathology of the monument.

The comparative examination of all available quantitative and qualitative data proved the inherent vulnerability of the structural system of the monument, in which (a) a stiff central cupola is resting (through the drum) on four major arches parallel to the two main

axes of the church, as well as on four arches oblique in respect with the longitudinal and the transverse axis, (b) the vertical, as well as the horizontal component of the self-weight of the whole system of (intersecting) groin vaults arranged around the central dome, are transferred to rather flexible stone masonry piers, (c) there are not ties in the arches and vaults, or other elements to confine critical structural parts or link the

various parts among themselves. Therefore, there is a tendency of the structural system to deform laterally in its upper region even under self-weight alone. Expectedly, this behaviour is deteriorating when a seismic event occurs, during which, according to the results of the preliminary dynamic analysis of the structure, out of phase movement of various parts of the monument occurs (Figure 21b).

Due to the inherent vulnerability of the structural system of the Katholikon and the high seismicity of the area, proved also by its historical pathology, several parts of the structure have been deformed independently; multiple cracks have emerged, dividing the entire structure in separate parts, and thus, interacting with nonlinear behaviour. Such behaviour can explain the severe damages occurred to the drum of the dome, as well as to the arches and vaults supporting the dome. As aforementioned, this ascertainment was also confirmed by the preliminary numerical analysis (Figure 21b) and reported in the first phase of the restoration study of the Katholikon [27].

On the other hand, such behaviour suggests that in addition to interventions that are necessary for the repair and strengthening of damaged masonry elements, adequate strengthening measures (in form of tie-rods, diaphragms over the extrados of vaulted roofs to connect the vertical walls, pillars and piers, bracing of the windows of the dome, confinements of drum and piers, etc.) should be taken. This conclusion is in accordance to the historical pathology of the monument and the fact that various parts that have been simply reconstructed in the past were cracked again (like the piers of the drum of the dome reconstructed in 1891 and cracked soon after, during the 1894 Atalanti earthquake)

or presented out of plane deformation (as for example the south wall of the narthex, already reconstructed in 1896 and deformed again, as measured after the 1999 earthquake).

Consequently, besides the repair and strengthening of masonry elements, a series of interventions to improve the overall behaviour of the structure were also schematically proposed (Figure 22), by the first restoration study, for further elaboration [8, 27]. The aim was to alleviate the inherent vulnerability of the structure and, thus, to improve its future behaviour in some critical regions, avoiding the danger of further deterioration of deformations and eventual local collapses.

The application of such measures was approved in principle by the Scientific Committee and the responsible authorities. Additionally, given the importance of the monument and the high values of its mural mosaics, the need for further research and investigations was also recognized.

5.3. Design in process for structural interventions optimization

Due to the high values of the monument and its vulnerability, the decision was taken to start the application in priority of a first series of interventions to achieve the best possible repair and strengthening of masonry elements, avoiding further deterioration of damage (which could lead to a total disruption of continuity and even local collapses) and in parallel to investigate thoroughly its structural behavior under seismic actions. The aim was to design the optimal interventions, considering all the values of the monument, and above all its mosaics, i.e., those interventions that achieve the required structural performance with the

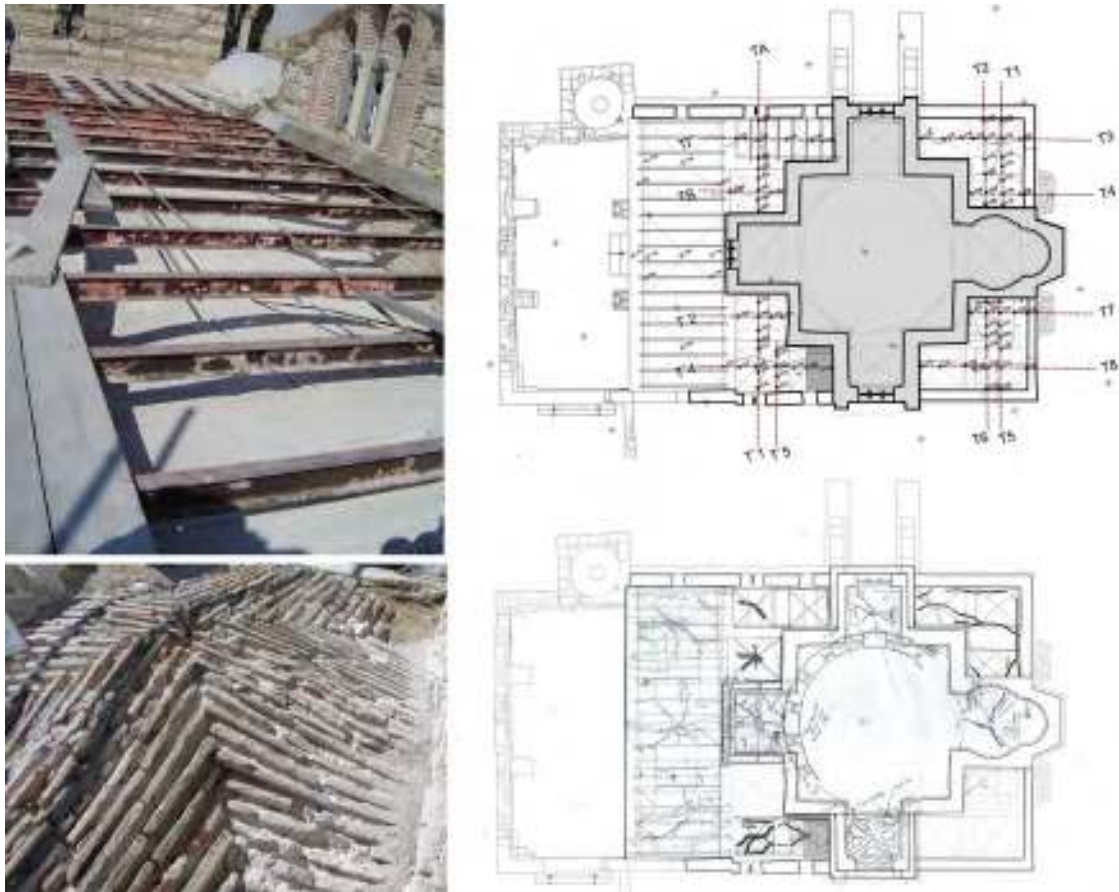


Figure 23. Survey of the geometry and the pathology of groin vaults extrados, of the steel beams used in past interventions and of the space available for future interventions [8].

minimum possible impact on heritage values, while ensuring the protection of the mural mosaics. To this end additional data were necessary to perform all necessary numerical analyses and safety checks with calibrated models and reliable data, avoiding conservative assumptions (both for actions and resistances). Such assumptions would lead to extensive interventions that might not be needed and that would inevitably alter the architectural value of the monument.

Thus, a design in process was adopted, a series of research and investigations were initiated and an earthquake monitoring system was installed in the monument, to collect real data for the dynamic characteristics of the structure, before, during and after interventions. This step by step multidisciplinary approach led to the partition of the design of the project and the implementation of the works in two phases, giving the possibility for the necessary additional data to be

collected and detailed structural analyses and safety checks to be carried out.

The study for the first phase of structural restoration interventions was completed in 2003 [4, 27]. The first phase of works (2004 – 2007) comprised all measures considered necessary for the repair and strengthening of masonry elements (stitching of cracks and deep re-pointing where necessary, systematic hydraulic lime grouting, local reconstructions, etc), and some local morphological restoration interventions, to correct alterations provoked in the past. The works of the first phase were applied in priority, in order to avoid further deterioration of damage, which could lead to a total disruption of continuity and even local collapses, in case of further seismic events [7].

In parallel with the works of the first phase, the following research and investigations were undertaken to support the design of both phases of interventions:

(i) Seismic monitoring of the structure for investigating its behaviour before, during and after interventions [10]. In this framework a special study for the estimation of the seismic hazard was also carried out; (ii) Design of adequate mortars and grouts [28-31]; (iii) Experimental investigation of the mechanical behavior of brick enclosed masonry bearing mural mosaics, before and after grouting [11]; (iv) Application of NDT's for investigating non-visible parts of the monument, with emphasis, on one hand, to the documentation and survey of the construction type of masonry [25] and of the mosaic's substrata [32, 33], and, on the other hand, to the control of the grouting effect and efficiency [32, 33]. A brief synthesis of all the main results of the afore-mentioned investigations is given in Miltiadou-Fezans 2008 [7].

Furthermore, detailed survey of the materials, geometry and pathology of many parts of the monument (Figure 23), that became accessible thanks to the realization of the works of the first phase, was carried out (e.g. internal faces of masonry walls and the intrados and extrados of vaulted roof).

The implementation of the first phase of structural restoration interventions has been accomplished in 2007, together with the most of the aforementioned research, surveys and investigations.

In this framework, all aforementioned data and research results obtained were used for preparing and calibrating adequate numerical models and perform structural analyses and safety checks. Moreover, all geometrical, structural and morphological data of visible and hidden parts were as well used for developing a 3D architectural model, in which all decorative elements (mosaics, frescoes, marble cornices, etc.) were as well included.

The design of the application project of the second phase restoration interventions of the Katholikon, including all necessary structural analyses, checks and calculations for the assessment of the efficiency and the dimensioning of optimum interventions, as well as detailed drawings, after in situ checks of their applicability (actual geometry, mosaics locations, etc.), was elaborated by a multidisciplinary working group in 2012 [34].

The second phase of works (2011-2016) comprised the strengthening interventions designed and selected as optimal to improve the overall structural behaviour of the building, including the installation of ties and struts, diaphragmatic structures over the extrados of the vaults and in the exonarthex, bracing of the windows in

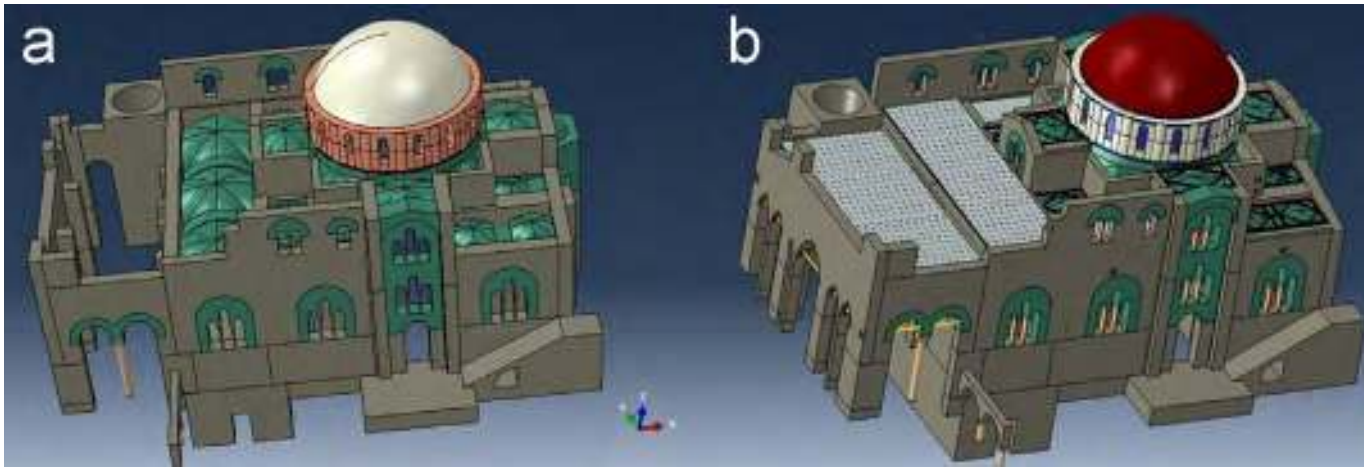


Figure 24. The numerical model of the entire monument 24a. without and 24b. with interventions adapted from [8] and [9].

the drum of the dome, confinement of structural elements, etc. The interventions of the second phase aimed to reduce the vulnerability of the structure and the extent of the damage under future seismic events, especially in the upper level, where the mural mosaics are located. In this second phase were also included all the works for the morphological, aesthetic and functional restoration of the monument.

As it is reported in more details in other papers [12, 13, 35], detailed numerical simulation and structural analyses of the Katholikon was carried out by means of more sophisticated tools, considering the improvement of the structural behaviour after the implementation of grouting, on the basis of the results derived from the afore-mentioned experimental research on masonry behaviour, before and after grouting. Two separate finite element models have been developed using solid finite elements (34, 12, 13). The first model comprised the hemispherical dome and its drum, while the second simulated the entire structure of the monument (Figure

24a). The numerical analyses were performed using the code ABAQUS (2011) and three-dimensional, tetrahedral finite elements. The separate drum-dome model was analyzed with seismic actions applied as equivalent static loads, while linear time history dynamic analyses were performed for the entire structure model. To calibrate both models, the data recorded by the monitoring system, were appropriately analysed and used. Their capacity to numerically reproduce the historical and recent pathology, identifying the most vulnerable areas of the monument, was also checked. The necessity of applying strengthening measures to alleviate the structure, improve the inherent structural drawbacks and reduce its vulnerability, already identified by preliminary analyses, was confirmed and further refined. Consequently, the calibrated models were used to qualitatively evaluate the effectiveness of alternative structural strengthening interventions (installation of ties, masonry confinement, diaphragms, etc.) and combinations thereof, aimed at improving the structural behaviour of

the monument. For the design and dimensioning of various strengthening measures, the models were adequately modified and, in some cases, “cracks” were induced, to overcome shortcomings of elastic analysis. After the evaluation of the efficiency of each alternative intervention, further analyses were performed for selected combinations of measures (Figure 24b).

Thus, the second phase of structural restoration works (2011–2016) comprised a combination of the following interventions: the installation of ties and struts; the construction of diaphragmatic structures over the extrados of the vaults and in the exonarthex; the bracing of the windows in the drum of the dome; and the circumferential confinement of various structural elements.

Apart from developing adequate numerical models to perform structural analyses, a 3D architectural model was also developed, based on all geometrical and morphological data of visible and hidden parts, including decorative elements. The structural and architectural numerical tools developed were complementary and necessary to design and dimension the various stainless steel or timber elements and their connections with masonry, considering the real geometry. Thus, the applicability of proposed measures was ensured and their effect on the appearance of the monument was fully documented.

5.4. Brief Presentation of the Structural Restoration Interventions

5.4.1. Masonry Repair and Strengthening Interventions

A series of interventions was applied to improve, in the

best possible way, the mechanical behaviour of masonry elements by a) reinstating their continuity, b) increasing their resistance (mainly to tension and shear), and c) ensuring that no side effects might occur, due to possible durability issues. The masonry repair interventions comprised mainly the following works (Figure 25, 26, 27): i) very careful removal of internal renderings and deteriorated pointing mortars applied during previous interventions, without harming the old ones, adjacent to or underneath them, ii) removal of tiles and other covering and filling materials to reach the extrados of all vaulted roofs, iii) stitching of the most severe cracks, using long stones, bricks, or thin titanium plates, iv) few local reconstructions, necessary either for the repair of dislocated or collapsed masonry parts or for the restoration of past morphological alterations, v) deep re-pointing where necessary, vi) preparation of masonry for grouting, vii) implementation of grouting, viii) removal of all injection tubes and filling of holes with mortar, (ix) in situ conservation of deteriorated old mortars, x) execution of all necessary works to ensure the protection of the extrados from rainwater. The application of adequate mortars and grouts compositions, constituted a key parameter in ensuring a successful intervention, both for the masonry and the mosaics. Thus, extensive laboratory research and in-situ pilot tests have been undertaken by the Directorate for Technical Research on Restoration (DTRR) of HMC to design the adequate mortars compositions, considering the characteristics of the existing materials, the requirements set by the restoration study and the worksite conditions. For most of the deep and surface re-pointing works lime-pozzolan based mortars were used. For the surface re-pointing of the façades old



Figure 25. First phase structural restoration interventions (removal of 1960's cement re-pointing mortars, stitching of cracks with titanium plates or stones and local reconstruction to correct past alterations) adapted from [9] © Hellenic Ministry of Culture.

pieces of ceramics from archaeological excavations were ground in situ in the appropriate granulometry and used in the mortar mixtures, to avoid the use of modern bricks fired in high temperatures. For local reconstructions, deep re-pointing of extremely damaged areas and crack-stitching using titanium plates, hydraulic lime-based mortars were applied. Detailed information on the characteristics of existing mortars is given in [19], while a synthesis regarding the old mortars and the basic compositions designed and applied to the monument are reported in [30].

The in-situ application of grouting had to be implemented without removing the old pointing mortars and, of course, without removing the mural mosaics decorating its interior (Figure 26). Consequently, due to the multitude of cracks, the grout for masonry was expected to penetrate not only to the masonry but also to the cracked substratum of the mosaics. Besides, the application of grouting was also proposed to improve the bond between mosaics substrata and masonry [36].

Thus, special attention had to be given to designing adequate grouts, not only because grouting constituted the main intervention undertaken to substantially improve the mechanical behaviour of the masonry, but

also because it is an invisible, and thus, irreversible intervention, that affects the masonry elements and all the mosaics, frescoes and old mortars borne by them. To this end, a set of design requirements related to the properties of the grout (namely, injectability, strength and durability) was determined, with the purpose of achieving a material suitable for both masonry and mosaics. Extensive experimental investigations were carried out by the DTRR and NTUA to design adequate grout compositions, to assess the mechanical properties of masonry elements, before and after grouting, and finally to evaluate the effect of grouting on their mechanical properties [11, 28, 29]. Furthermore, as the proper design of a grout composition cannot, on its own, ensure the successful execution of the grouting intervention, an application methodology was developed [31] for the implementation of injections in a more rational and fully controlled way, using entrance and exit tubes of small diameter in areas with mosaics and old mortars (Figure 27), low pressures and detailed quality control of the grout and the injection process to ensure that the grout reached and filled the numerous cracks and fissures.

The application of grouts to masonry bearing mosaics being a delicate procedure, the decision was made to



Figure 26a. North external façade of the East arm of the cross bearing old byzantine mortars (left) and the heavily damaged mosaic in its internal façade (right). 26b. External old mortars of 1st and 2nd construction phase (left) and internal precious mosaics, which had to be preserved in situ (right) adapted from [9] © Hellenic Ministry of Culture.



Figure 27. Preparation for grouting of masonry, bearing old mortars and of mural mosaics adapted from [9] © Hellenic Ministry of Culture.

train the Personnel through trial applications. To this end, in addition to the Personnel of DTRR and NTUA, the team of experienced Conservators of the DCAMM, working at Daphni Monastery, was also involved in the experiments carried out at the Laboratory of the DTRR and the LRC of NTUA, contributing to the construction of mosaic's replicas on the wallettes, to the installation of grouting tubes on the mosaics, and to the implementation of injections. The novel methodology was tested and further refined, during the injections of the specimens made for in-laboratory testing. Thus, the in-laboratory work served as a full-scale rehearsal of the entire grouting procedure on historic masonries bearing precious mosaics. Furthermore, this was the best way for training the in-

involved Personnel. In this way, collaboration and osmosis were developed within the multidisciplinary group. The mutual understanding established led to a smooth and fruitful cooperation, between those working on masonry and those working on mosaics, during the in-situ implementation of grouting, necessitating meticulous and refined interventions. For instance, plastic transparent tubes ($d=4-10\text{mm}$) had to be installed on both faces of masonry and at adequate distances, forming a grid ($l\sim 0.3$ to 0.8m , both horizontally and vertically), together with similar, but finer, tubes ($d=1.0-3.3\text{mm}$) installed in the mosaics. Moreover, various compositions of lime-pozzolan or hydraulic lime mortars had to be prepared and applied carefully to seal voids, fissures, cracks and detachments,

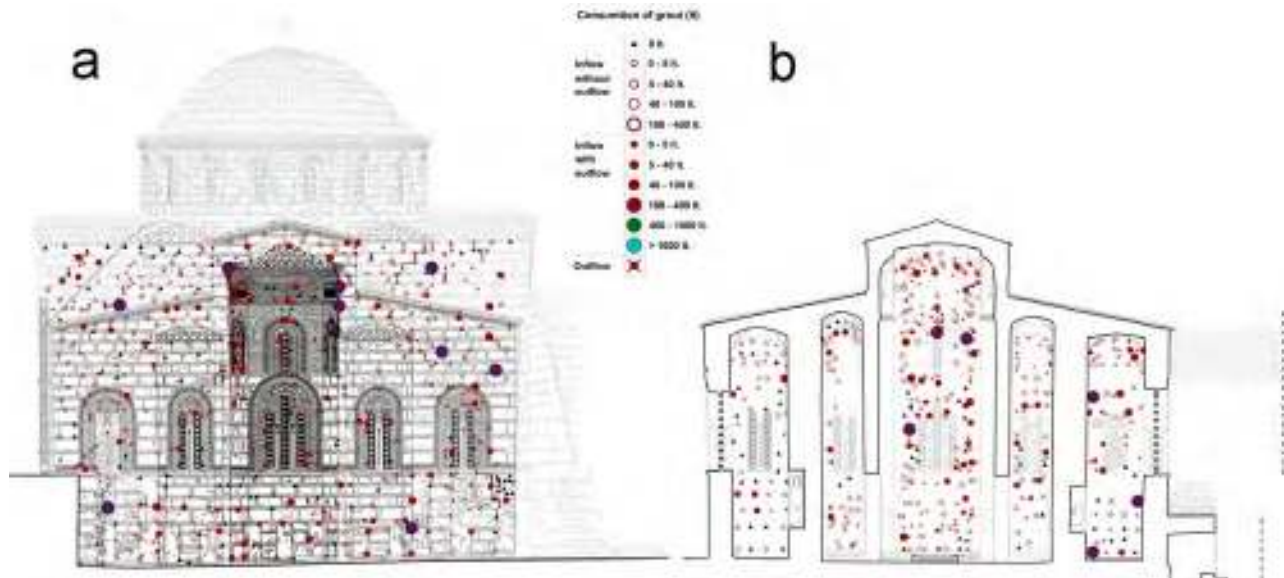


Figure 28. Schematic presentation of grouting consumption on the a) external and b) internal east façade adapted from [9].

while the tubes were inserted in various depths, with the purpose to reach the interfaces between the various layers of mosaics substrata, as well as interfaces between masonry leaves [31].

Besides, all this information, along with the number allotted to each tube, had to be reported on drawings (Figure 28, 29). Throughout the whole project, entrances and corresponding exits of the grout were recorded on a time basis, together with any changes of pressure at the entrance to the wall. All these data were noted on specially designed daily calendars, with reference to the area of grouting application. These manually collected data and those collected by the Grout Recorder (pressure, flow rate and grout volume consumed) were then combined, and the grout volume corresponding to the various groups of entrances and exits was determined, together with the total volume

of the grout consumed during the corresponding day. All the recorded data were organized in tables and after appropriate processing the volume of the grout consumed into the masonry was extracted. Then the data collected were reported on the series of drawings presenting the positions and numbers of the grouting tubes. Thus, a good estimation of the grout movement and consumption in relation to the various regions of the structure was achieved. This procedure was followed throughout the project in the Katholikon, both for the masonry and for the mosaics. In Figure 28 the corresponding drawings for the internal and external façade of the east wall of the monument are presented. One can easily recognize the areas with high grouting consumption, and those with a lower one. The tubes with no consumption are also noted. Thus, for this specific wall the volume of the grout consumption was

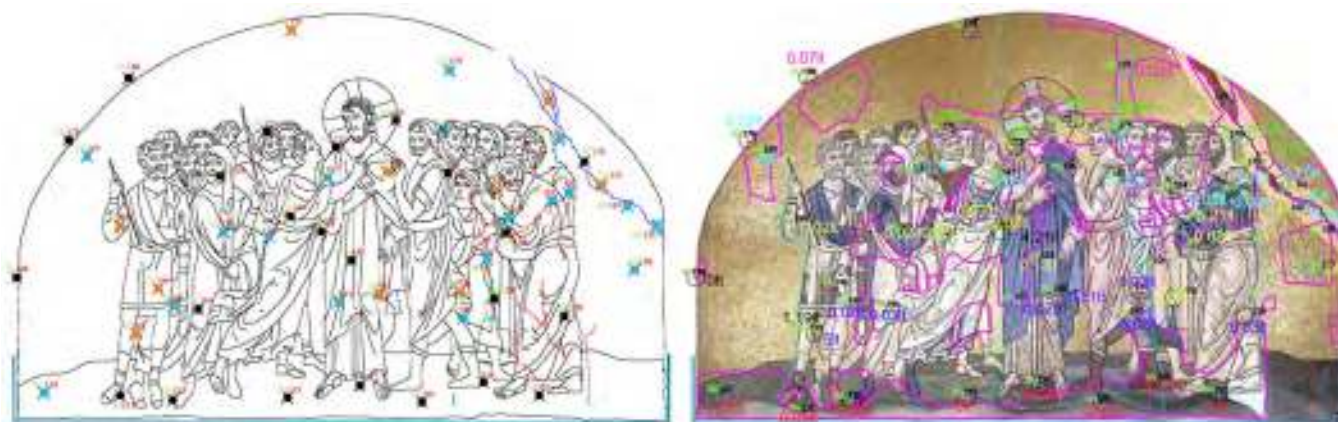


Figure 29. Juda's Betrayal mosaic: Survey of injection tubes (left) and of grout volume consumed (right) adapted from [9].

estimated to reach 6.5% of the total volume of the wall. In the case of the west wall of the monument, which was reconstructed during past restoration interventions (Delinikolas et al. 2003), the estimated grout consumption attained 2.5% of its total volume [31]. Using this methodology, the movement of the grout inside masonry and behind the mosaics was continuously monitored (Figure 29). Thus, both for masonry and mosaics, it was checked that the grout reached and filled the cracks and voids, ensuring the quality of the intervention. The seismic monitoring data and those obtained by sonic tomography [10, 33] carried out during and after the grouting, provided valuable information for the effect and the efficiency of the overall grouting intervention, and enhanced the validity of the methodology applied.

5.4.2. Strengthening Interventions Improving the Overall Behaviour of the Monument

In this paragraph, a concise synopsis, per area, of the strengthening interventions, selected as optimal to

improve the overall behaviour of the structure, taking into account all the values of the monument and above all the protection of the mosaics, is given:

- In the dome and its drum: Replacement of the existing confining steel ring-plate, situated at the upper level of the drum, by a stainless steel one. Installation of a second non-visible confining stainless-steel ring in the extrados of the hemispherical dome, underneath the ceramic tiles. Insertion of stainless-steel stiffening frames in the windows of the drum (Figure 30b, 30c, 30d).
- Over the extrados of the groin-vaults of the main church: Installation of stainless steel "diaphragmatic" structures; their final geometry was adjusted to the empty space existed in each area (Figure 30a, 30b).
- Over the extrados of the groin vaults of the narthex and W chapels: Installation of a diaphragm composed of LVL wooden plates (kerto Q) supported by stainless-steel beams (for the vertical loads) and stainless-steel beams at the perimeter, connecting the entire system to the walls (Figure 30a, 30b).



Figure 30. The optimum strengthening measures, as presented in various images of the 3D model adapted from [9].

d) In the exonarthex: Installation of a diaphragm composed of LVL wooden plates (kerto Q) supported by wooden beams (for vertical loads), and stainless-steel perimeter beams, connecting the entire system to the walls. Installation of stainless-steel ties and wooden struts in the arches and replacement of the masonry piers, built by Stikas (1962), with copies of marble ionic columns (Figure 30a, 30b, 30c, 30e).

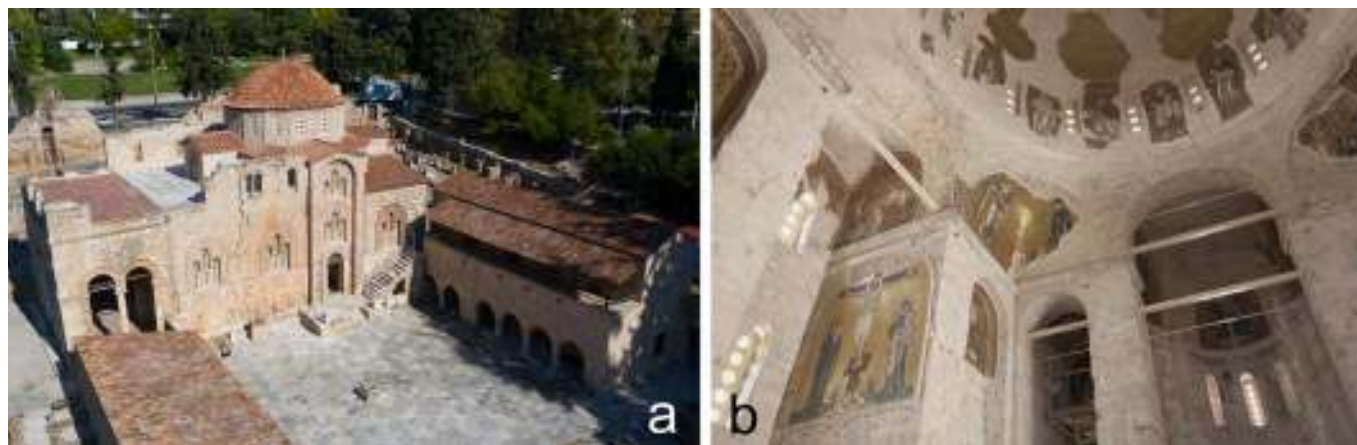
e) In the interior of the main church, the narthex and

the crypt: Installation of stainless-steel ties and struts in most of the arches. Confinement of west piers of the church and the piers of the crypt, with stainless steel plates (Figure 30e).

f) In the tower of staircase: Confinement of the masonry walls in three levels, by insertion of titanium threaded tie-bars, bonded to masonry by means of hydraulic lime injections and anchored externally on titanium anchorage plates. Installation in the tower's interior of a round steel staircase (Figure 30d, 30e). As aforementioned, the application of the works of the second phase has been accomplished in 2016. In Figure 31a an external view of the restored monument from the southwest is given. An internal view of the N-E part of the monument is presented in Figure 31b.

6. Conclusions

The brief presentation of the restoration the Katholikon of Daphni Monastery attempted in this paper, highlights the variety and complexity of the problems and the importance of a holistic approach, of synchronized transdisciplinary efforts at various levels, not only technical and scientific, but also administrative and economic, in order to find the optimal solutions in each step, while organizing in the same time the next step. The design in process adopted for the restoration of the Katholikon proved to be very efficient, as it gave the possibility to perform the in situ and laboratory investigations that are necessary for the design of the next step, as well as for the evaluation of the previous ones. Based on ISCARSAH recommendations, adequate in-situ and in-laboratory investigations and research were carried out. Novel methodologies, seismic monitoring procedures, innovative NDT's and intervention



Figures 3 1a. External view of the restored monument from SW and 3 1b. an internal view of the NE part © Hellenic Ministry of Culture.

materials and techniques, as well as digital modelling, were developed for the rational design of the optimal interventions taking into account all the values of the monument and aiming to improve its seismic behaviour. Thus, the implementation of the aforementioned optimal structural restoration interventions aimed to modify the static and dynamic behaviour of the monument only at a level necessary for strengthening the structure, without radically altering its authentic existence and the integrity of its form and without any essential impact on the existing mural mosaics. The behaviour of the structure in future earthquakes was improved, without excluding however, even for the level of seismic actions considered, the appearance of damage in case of a strong earthquake.

Furthermore, most of the interventions were designed to be reversible so as to give future generations of architects and engineers the possibility of re-intervening in the monument, probably by means of more advanced techniques.

In 2016, seventeen years after the 1999 Athens earthquake, the structural restoration of the Katholikon has been accomplished.

It must be noted however, that in such important monuments, preventive restoration and conservation interventions are always necessary and should be considered in a maintenance plan.

Last but not least, it is strongly believed that the experience and knowledge acquired by all levels of the Personnel involved in the framework of the structural restoration of this important monument of World Heritage constitute a solid base for research, investigations, studies and structural restoration interventions on other similar historical structures and monuments.

Acknowledgements

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Writing an Explanatory Report

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1. Introduction

How do you communicate the findings and recommendations from a multidisciplinary team of specialists so that it can be clearly understood? The Explanatory Report is a synthesis of the findings and recommendations of the various members and experts of a multidisciplinary team. In many instances extremely complex data must be interpreted and explained in a manner that can be understood by the layperson. This is one of the greatest challenges of the conservation engineer – communicating their findings with others not in the engineering discipline. Thus, the goals of the explanatory report are the following:

1. The decisions on any proposed intervention must be clearly communicated to laypeople: owners, managers, stakeholders, etc.
2. In a multidisciplinary team (Figure 1), there may be many writers who may not share the same priorities; and will not share the same writing styles, terminologies, etc. However, the completed report must be organized to present priorities within a set outline.



Figure 1. For the 192-meter-tall Gateway Arch project in St. Louis, Missouri, USA a multidisciplinary team was developed that included a corrosion expert, stainless steel expert, engineer, architects, architectural conservator, chemists and geologists, owner's representative, difficult access expert, contractors to provide access, and safety officer. Many of the members were tasked with writing portions of the report which needed to be edited for standard alignment to speak "one tongue."

Therefore, a homogeneous writing style and use of terminology must be adopted.

3. Qualitative judgements, cultural and socio-economic constraints may play a role as important as the quantitative data obtained by means of the investigation.

Such constraints must be understood and discussed within the report. That is not to say that economic factors will drive decisions on safety and durability.

4. It is essential that any assessments of safety and possible recommendations for structural intervention are set out and written in a form that is understandable to non-engineers, so that they can be considered together with cultural, social, political, and economic factors.

II. What may be included in the Explanatory Report

In summary the explanatory report should present a critical analysis of the steps taken and an explanation of the decisions throughout the diagnosis and intervention phases. Heritage buildings and structures are unique and were not designed or constructed in accordance with standards established for contemporary structures.

A report on findings may need to be customized to fit in with the particular building. The explanatory report should, as a minimum, include the following components:

Diagnosis

Diagnosis includes the investigation to identify or determine the nature and causes of deterioration and decay, and the opinion derived from such an investigation (Figure 2).

The following should include:

1. The hypothesis or hypotheses to describe the cause of the building conditions.



Figure 2. At the Tangier American Legation Museum in Morocco a sondage was required to expose the foundation systems as part of diagnosing issues with building settlement.

2. A categorical presentation of investigative techniques and their findings. These investigative techniques are typically discussed from the most basic to the more sophisticated.

An example of the spectrum of these techniques might include: archival research; geometric documentation; nonintrusive survey; intrusive survey; field and labo-

ratory examination of removed materials and components; mathematical calculations with or without the use of computers; and physical health monitoring.

3. The reliability of the data that is collected and any uncertainties should be presented.

Uncertainties may include, among other aspects:

- a.** The nature of the structural scheme.
- b.** Knowledge or lack thereof of any inherent structural weaknesses.
- c.** The properties of the materials.
- d.** The nature of static, dynamic or chemical/biological forces acting upon the structure.

4. The diagnosis of the causes of material decay and structural damage and the evaluation of the overall condition of the structure.

5. Proposals for further essential investigative work that were included in the original investigation but became apparent based upon the findings to date. Such further investigative work may include continued observation work, further laboratory studies and long-term monitoring.

Therapy

Therapy includes remedial measures such as emergency measures, maintenance, conservation, preservation, rehabilitation, strengthening, reconstruction, etc. in response to the diagnosis (Figure 3).



Figure 3. As part of the therapy on the 121 meter tall Nebraska State Capitol, USA the limestone units were selectively removed to expose the steel support structure beneath so that the corrosion could be removed, the steel painted, and flashed prior to reinstallation of the limestone.

The following information should be contained in the report:

- 1.** If the building does not meet structural requirements, justify the intervention needed to improve it. Information should be provided on the approaches

utilized for structural verification and how they have led to these conclusions.

2. As any therapy enacted will have some unintended effect, it is best to minimize the intervention as much as possible.

3. A detailed description of the selected intervention. In specific, any proposed intervention must consider the temporary situation during the execution of the works when there may be a higher level of risk. Such situations appear frequently in structures that have already suffered considerable damage, as in post-earthquake works.

4. Provide a description of the alternative intervention solutions proposed.

The report should consider the pros and cons of each alternative and discuss their compliance with the structural requirements and their impact on the structure's cultural value. Such a detailed analysis justifies the final proposal and provides the best guarantee of the quality of the project.

a. Provide a description of the adopted performance criteria. If the performance criteria is reduced with respect to the application of current design codes to limit excessive intervention, justify this decision and explain how adequate safety is provided by alternative means.

b. Prioritize the therapies that are recommended in the following manner: emergency works works that are urgent; works that are essential for both the im-

mediate and long-term stability; and works that are desirable to either mitigate the level of maintenance or improve reliability.

III. Organization of the report

The organization of the explanatory report is a matter of personal preference. However, to help the author, the following comprehensive, much used and successful report organization is provided as a starting point:

- ***Executive Summary*** (typically the last part of a report to be written but put at the front of the report to summarize findings and recommendations)

- ***Introduction***

- Definition of the object and the need
- Introduce the players (investigators, owner, stakeholders, etc.)
- When was the work carried out

- ***History and Description (technical description)***

- ***Methodology***

- Archival studies
- Geometric documentation
- Non-intrusive observation
- Intrusive observation
- Materials studies
- Computational analysis
- Long term monitoring

- ***Discussion (critical analysis and uncertainties)***

- ***Conclusions***

- Reliability of data

- Future investigative proposals

- **Recommendations**

- Intervention options and recommended intervention
- Priorities

- **References**

- **Annexes** (*place full reports of members of the team that have been paraphrased in the main text of the report*).

IV. Strategies for complex report writing

The following strategies for complex report writing are provided by the author to clearly convey the findings and recommendations:

1. Define the orientations of the building or monument along cardinal directions and with respect to streets and primary facades.

Use these orientations throughout the report.

2. Define technical terminology to describe the building or monument and use them exclusively. If appropriate, provide this terminology with

definitions in a footnote or glossary.

3. Let the report speak with “one tongue.” The best method to accomplish this task is to allow a single person to edit the entire report rather than patching together sections of the report that are written by various members of the multidisciplinary team.

4. When possible, provide data and interpretations in graphic form. Successful graphic representations can easily be understood without the specific need for language skills.

5. Keep photographs, illustrations, graphs, charts and the like near their citations in the text. Report reading is cumbersome if flipping back and forth between the text and figures is required.

6. Develop chapters with a new numbering system for each chapter and a complementary numbering system for photographs, illustrations, graphs, charts and the like.

In this manner if editing of a particular chapter is required the enumerations or the other chapters will not need to be revised.

Mechanics of Historical Masonry, Materials

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Abstract

In this paper, the variety of historical masonry types of construction is presented, along with basic mechanical properties. The difficulties encountered when assessing the behaviour of various types of masonry are presented, as well as the lacunae in the available data. To reach sufficient knowledge on the current state of the structure-a prerequisite for its assessment, as well as for the selection of adequate intervention strategies-exhaustive documentation of historical structures and monuments is needed. Several examples are presented to support this need.

Keywords: Dry construction, brick masonry, stone masonry, earth masonry, mortar, mechanical properties, documentation, in situ investigations

1. Introduction

Masonry is a general term, covering a vast variety of materials combined to create numerous alternative construction types (Figures 1 and 2): Dry construction, various types of stone masonry (single-, double- or three leaf) ranging from rubble to cut stone masonry, brickwork, to form walls or arches, vaults and domes, as well as various types of earth masonry.

On the other hand, depending on the area and the era, various types of historical mortars can be identified (ranging from earth mortar, to lime and lime/pozzolan mortar, as well as to mortars containing gypsum-frequently used in Iranian monuments). The Structural Engineer, in need of data regarding the mechanical properties of masonry of the historical structure that is to be assessed and repaired or strengthened, remains perplex facing this multitude of forms in which the material “masonry” appears in cultural heritage assets. This is because the basic mechanical properties of masonry (such as compressive [1], shear, flexural strength of masonry, as well as deformation properties) are expected to depend strongly not only on the characteristics of the constituent materials but also on the type of construction of masonry on its faces and within its thickness. The question arises whether the data needed for the estimation of the mechanical properties of so many different types of masonry are available. This is an issue of major significance, as the adoption of conservative values for the mechanical properties of masonry is not an option in case of monuments and historical structures. In fact, the underestimation of the available

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Figures 1. Various types of construction of masonry faces: dry-, brick, stone- and earthwork.





Figures 2. Various types of construction within the thickness of masonry.

margin of safety may lead to unnecessary and invasive interventions that alter the Values to be preserved. In the following sections of this paper, the behaviour of masonry is discussed upon, the available knowledge is presented in brief, whereas means to obtain data that may assist the Structural Engineer in the effort of assessing the current state of a historical structure are presented.

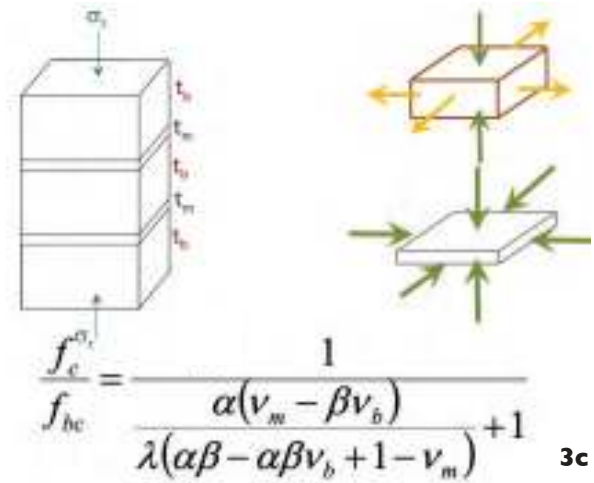
2. The behaviour of masonry in compression

2.1. Single-leaf masonry in compression

Masonry (Figure 3a) is known to be a composite, anisotropic and brittle material. It is indeed made of two or more materials (masonry units, mortar, timber,

iron, etc.), its mechanical properties differ depending on the direction of loading, whereas it reaches its maximum resistance at relatively small deformations; a rather steep falling branch follows the attainment of maximum resistance.

Testing of masonry in compression shows that failure is due to the occurrence of vertical cracks, both on the faces of the wall and within its thickness (Figure 3b). The cracks cross mortar joints, as well as the much stronger masonry units. It should be noted that, in the tests shown in Figure 3b, the compressive strength of the bricks was equal to 50MPa, whereas the compressive strength of the mortar was equal to 1MPa only. The working hypothesis adopted with the purpose of interpreting such a behaviour is the following: As the modulus of elasticity of mortar is



Figures 3a, 3b, 3c. The Mechanics of single leaf masonry to compression.

significantly smaller than that of the masonry units and-on the contrary-the Poisson's ratio of the mortar is higher than that of the masonry units, the vertical stress, σ_z , imposed to the masonry prism (Figure 3c), leads to higher lateral dilatancy of the mortar than the brick. As the lateral deformation of the prism at the level of mortar joints has to be equal for the two bonded materials, the mortar exerts horizontal tensile stresses to the brick, whereas the brick exerts horizontal compressive stresses to the mortar. Thus, the mortar is subjected to (favourable) triaxial compression, whereas the brick is subjected to (unfavourable) heterosemous state of stresses. As a result, the tensile strength of the brick (under simultaneous vertical compression) is significantly reduced and this phenomenon may interpret the occurrence of vertical cracks crossing the strong material. On the contrary, the strength of the mortar in the joints under vertical compression is enhanced, thanks to the presence of

the lateral compression. Therefore, the compressive strength of masonry is expected to be higher than the uniaxial compressive strength of the mortar and lower than the uniaxial compressive strength of the masonry units.

This oversimplified model, based on the (obviously, unrealistic) assumption of elastic behaviour of the constituent materials of masonry, leads to a formula (Figure 3c), in which the ratio between the compressive strength of masonry and that of masonry units (f_c and f_{bc} respectively) is expressed as a function of the Poisson's ratio of the two materials (v_b and v_m), the ratio between the moduli of elasticity of mortar and brick (β), the ratio between tensile and compressive strength of the unit (λ) and, finally, on the ratio between the thickness of the mortar joints and the height of the masonry units ($t_m/t_b = \alpha$). Although oversimplified, this formula is able to predict with sufficient accuracy experimental results obtained from

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Figures 4. Typical historical masonries in Iran.

testing single-leaf brickwork [2] and [3]. Nevertheless, one has to admit that (a) in most of cases, the Engineer does not have access to detailed data on the mechanical properties of the materials (which, on the contrary, are characterized within research programs), such as Poisson's ratios. Furthermore, (b) the behaviour of masonry is highly non-linear and it cannot be accurately predicted on the basis of simple elastic models, and (c) historic masonries can hardly be simulated by the simple pattern of modern brickwork. It is interesting to observe that, even for modern masonries, Eurocode 6 [4] proposes an empirical formula for the calculation of the compressive strength of masonry.

In that formula, the compressive strength of masonry is expressed as a function of the compressive strength of the two materials, whereas an empirical factor is provided to allow for the type of both masonry units and mortar to be taken into account. It is noted that the prerequisite for the applicability of

the EC6 formula is that the thickness of the mortar joints does not exceed 15mm, whereas the thickness of masonry is equal to the width or the length of the masonry units.

Therefore, it is clear that EC6 cannot be applied to the majority (if not to all) of historical masonries, in which-typically-the mortar joints are thicker than allowed by EC6 or to multi-leaf masonries (Figure 4). In an effort to cover the need for estimating the (basic) property of historical masonry, i.e. its compressive strength, several empirical formulae are proposed in the Literature. As an example, the formula proposed by Tassios and Chronopoulos [5] is mentioned here (Figure 5).

This formula was applied to predict the experimental values of compressive strength of masonry found in the Literature [6].

Taking into account the scatter of the experimental results themselves, the formula is able to predict quite accurately the experimental values. It should be

$$f_{mc} = \left(\frac{2}{3} \sqrt{f_{sc}} + k_1 f_{sc} - k_2 \right) : \left[1 + 3,50 \left(\frac{V_m}{V_v} - 0,30 \right) \right]$$

V_m, V_v : the volume of mortar, the volume of masonry ($\geq 0,3$)

f_{sc} : compressive strength of masonry units ($< 100 \text{ MPa}$)

f_{sm} : compressive strength of mortar

$k_1=0,6$ (rubble stone masonry), $0,2$ (brickwork or cut stone masonry)

$k_2=0,0$ (brickwork or cut stone masonry), $0,5 \text{ MPa}$ (partly cut stone masonry), $2,5 \text{ MPa}$ (rubble stone masonry)

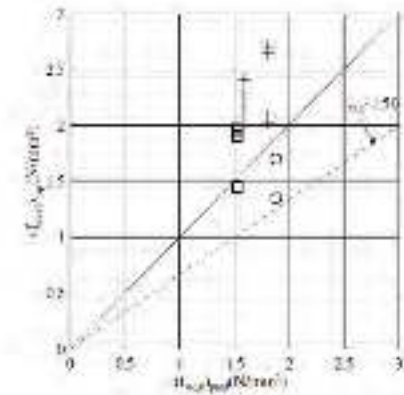


Figure 5. Empirical formula for the estimation of the compressive strength of historical masonries [5] and comparison between predicted and experimental values of compressive strength [6].

noted though that this formula cannot cover the quite frequent case of multi-leaf masonries.

2.2. Three-leaf masonry in compression

Three-leaf masonries constitute a quite frequent type of construction in historical structures, presenting a notable variety. Actually, in this type of construction there are two exterior leaves (in the usual case, following different construction patterns and of different thickness) and a core that may be solid or loose (consisting of small pieces of stones and/or bricks and mortar), the latter being the most frequent case (Figure 6). Furthermore, the exterior leaves are usually unconnected in the transverse direction (i.e. there are no headers providing their interconnection).

Three-leaf masonries present the following peculiarities, when subjected to compression: (a) In case of stone masonry, the two exterior leaves are not constructed according to the same pattern. The exterior leaf—that in many cases remains unplastered—is made of larger dimension semi-cut stones, whereas the in-

terior leaf (usually, plastered or painted or covered with frescoes or mosaics) is made of smaller rubble stones, whereas (b) the exterior leaves are not of equal thickness, (c) the roof or the dome/vault transfer their weight, as well as the horizontal component of their thrust to the interior leaf of masonry alone. Furthermore, (d) the bond between the two exterior leaves (ensured by the weak filling material) may be broken either due to decay or due to mechanical actions imposed during the lifetime of the historical structure. As a result, the two exterior leaves may be separated from each other.

Let us examine the effect of those peculiarities on the behaviour of three-leaf masonries: (i) The difference in construction types (and, perhaps, also the difference in thickness) of the two exterior leaves is the cause of an inherent eccentricity of the masonry element. Indeed, even if a vertical load is applied centrically to the compressed masonry, as it was the case in the specimen shown in Figure 7a, the two exterior leaves carry different portions of it, depending on their

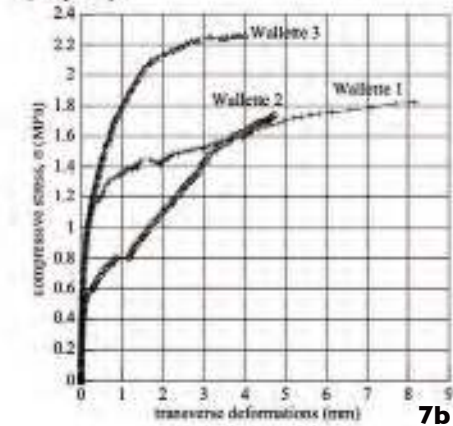
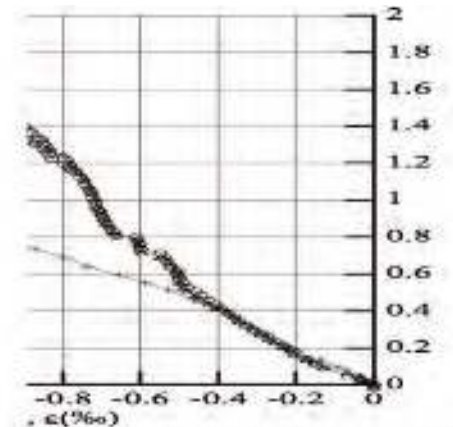
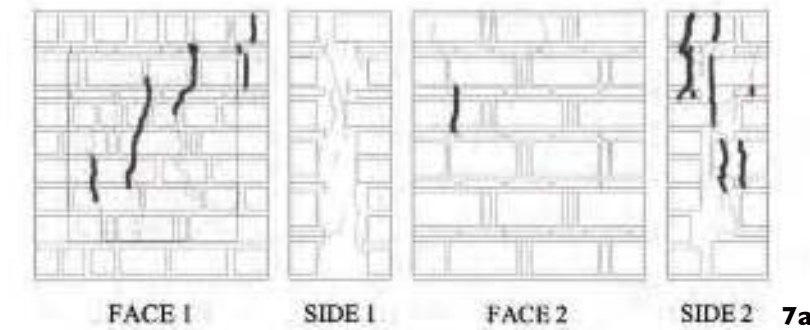


Figures 6. Three-leaf masonries in: 6a. Iran [6] and 6b. Greece.

stiffness and strength, which are different, although the materials used for their construction may be identical. Thus, the crack pattern of the two leaves is different, with the more extensive cracking occurring to the (typically, less strong) interior leaf (Figure 7a), which may host valuable paintings or frescoes or other decorative elements to be preserved. Furthermore, (ii) even if the bond between the two exterior leaves is active (as it was the case in the newly constructed specimens tested at the lab), one may observe that vertical cracks within the thickness of masonry, separating the leaves from one another, occur. Such a separation becomes imminent when there are curved elements (Figure 7c), of smaller thickness than masonry and typically connected to the interior leaf of masonry exclusively. In that case, as there is a tendency of part of the vertical load of the interior leaf to be transferred to the exterior one, (vertical) shear cracks occur at the interfaces between the exterior leaves and the filling material. (iii)

The result of the separation of the leaves is that the slenderness (say, the ratio between the height and the thickness) of the element increases significantly. If the assumption is made that all three leaves are of equal thickness, then-after separation-the slenderness of each individual leaf is the triple of the “monolithic” masonry. Thus, the failure of masonry is due to the buckling of the exterior leaves that are subjected to an eccentric vertical load. Such a failure mode was fully confirmed by laboratory tests.

Actually, in the two graphs of Figure 7b, one can see the horizontal deformation of the wallette shown in Figure 7a (the horizontal deformation stands for the sum of openings of all vertical cracks on the faces of the wallette), as well as the transverse deformation (which stands for the sum of the openings of all vertical cracks within the thickness of the wallette) vs. the imposed vertical load. One may observe that, although both families of cracks-those on the faces and those within the thickness-occur more or less at the same



level of loading (approximately equal to 1/3 of the maximum load), from then on, the cracks within the thickness open much faster than the cracks on the faces of masonry. Indeed, at the point of attainment of the maximum load, the total opening of cracks on the faces is of the order of 1,0mm, whereas that of cracks within the thickness is 5 to 6 times larger. Thus, this tendency of the individual exterior leaves to bend outwards is confirmed, although the tested wallettes were of very limited slenderness. The question arises whether those in-laboratory observations are confirmed also in-situ.

Figures 7a. Crack pattern of a wallette tested in-lab [7], 7b. deformation vs. vertical load for wallettes tested in [7], 7c. the cupola of the Katholikon of Dafni monastery supported by arches and squinches, 7d. cracks within the thickness of masonry walls after the 2016 Rieti earthquake, Italy.

As shown in Figure 7d, such a separation occurs in some cases as a crack between the leaves of masonry, in others in a rather catastrophic form. What is also of significance for the Structural Engineer who is responsible for the assessment of a historical structure at its current state is that the (critical for the bearing capacity of the element)



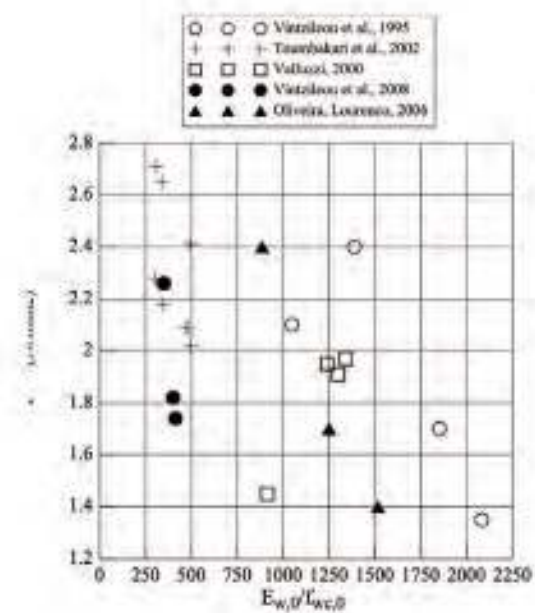
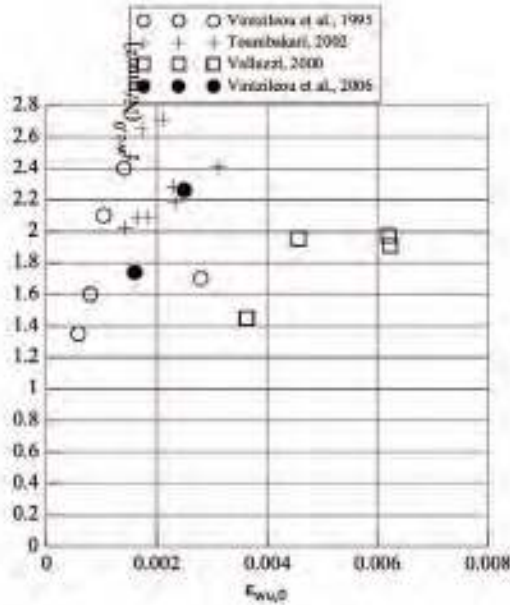
Figures 8. The effect of separation of the leaves of masonry: 8a. In-plane shear and partial out-of-plane collapse of a three-leaf wall and 8b. Out-of-plane collapse of the central region of a wall.

vertical cracks within the thickness of the elements are not visible from outside.

Therefore, the Structural Engineer, being adequately educated in the Mechanics of historic masonries, will have to check (e.g. through boroscopy) the inner state of masonry and detect cracks and voids within it. The consequences of non-taking into account the cracks within the thickness of masonry are shown in the pictures of Figure 8: Considering, during the assessment of the current state, the wall of Figure 8a as monolithic, its resistance to in-plane shear and/or to out-of-plane bending may be calculated to be sufficient for the expected seismic actions.



However, due to the separation of the leaves, the shear resistance of the weakest leaf is governing. The combination of in- and out-of-plane seismic action led to the partial collapse of the exterior leaf. In fact, the exterior leaf is subjected to smaller vertical load (as the roof is supported mainly by the interior leaf) and, hence, it is more vulnerable both to in-plane shear and to out-of-plane bending.



Figures 9a. The vertical strain corresponding to the compressive strength of three leaf masonries and 9b. the ratio of the initial modulus of elasticity and the compressive strength of three-leaf masonries plotted against the compressive strength [5].

Similarly, the separation of the leaves in Figure 8b led the wall to out-of-plane collapse at the region where the out-of-plane deformations take their maximum value. The Mechanics of three-leaf masonries been interpreted, the question is whether there are formulae (be it empirical) for the estimation of the compressive strength of those masonries.

Unfortunately, three-leaf is a quite general term for masonries. The systematic documentation of historic masonry buildings in various Italian regions [8] shows that in three-leaf masonry, usually, the ratio between the thickness of each exterior leaf and that of the infill is approximately equal to 1:0,50 [9]. The survey carried out by the Politecnico of Milan, together with the evaluation of the data reported in [10], led to the following geometrical data for three-leaf masonries:

Percentage of stones/mortar/voids: 55-85%/12-36%/0,4-15%. It is obvious that, even if those masonries were made of exactly the same materials, their compressive strengths would result significantly different. Actually, according to the evaluation of experimental data and in-situ measurements [10], the compressive strength of three-leaf stone masonry varies between 0,60MPa and 2,40MPa.

The deformation properties of historic masonries are quite scattered as well. As reported in [12] and [13], the strain corresponding to the compressive strength of single- or three-leaf masonry may vary between 0,20% and 0,80%. The evaluation of experimental results plotted in Figure 9a, show similar values [6]. Equally large scatter is observed in case of the modulus of elasticity of historical masonries (Figure 9b).

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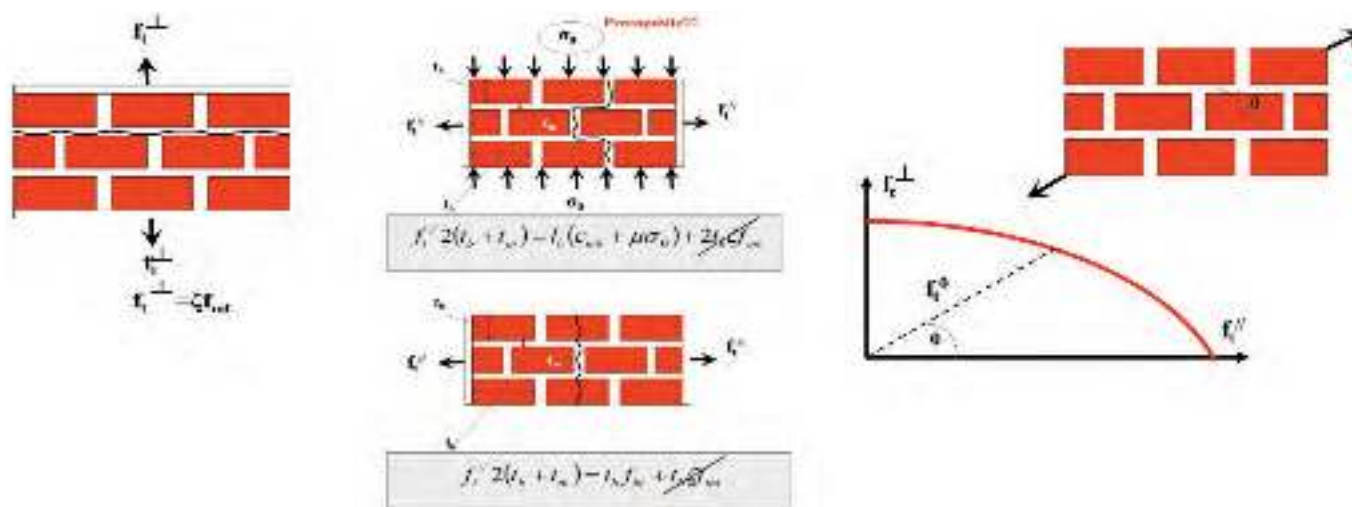


Figure 10. The tensile strength of masonry: 10a. Under vertical tension-failure occurs within the mortar joint or at the interface between mortar and masonry units, 10b. Under horizontal tension-the crack may cross masonry units as well, depending on the relative mortar-to-unit tensile strength, 10c. The dependence of the tensile strength on the direction along which tension is applied.

It seems, therefore, that when data representative of a specific type of masonry are needed, the available experimental results are not sufficient. In such cases, an alternative to laboratory tests and to the application of empirical formulae (whenever available for the construction type under examination), would be to perform in-situ tests on masonry.

However, it seems that this is a rather costly and time consuming alternative.

It may be a sensible solution either in case of an important monument or in case such tests are carried out in the framework of a study concerning, for example, an entire historic centre.

As a conclusion, one may say that the evaluation of the available data show that (a) the experimental results (from in situ and in laboratory tests) are limited to few types of historic masonry, (b) there is no general physical model describing the behaviour of

historical masonries in compression, not to mention that (c) it is quite uncertain to predict the elastic modulus of elasticity, as well as the deformation at failure of masonry in compression.

Last but not least, during a seismic event, masonry is subjected to oblique (changing in magnitude) compression with simultaneous transverse (also changing in magnitude) tension. Experimental results for this frequent case are not available for historical masonries. Relevant physical models are missing too.

3. Masonry in tension

The tensile strength of masonry is an unreliable property, depending on numerous parameters, such as the tensile strength of the mortar, the bond between mortar and masonry units, the curing conditions of masonry, as well as the direction in which tension is applied (Figure 10).

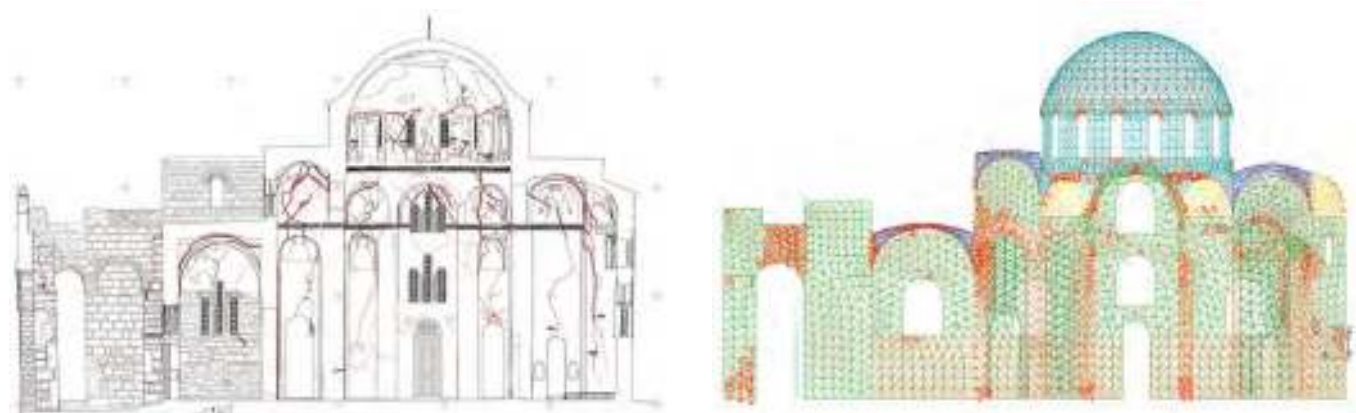


Figure 11. The Katholikon of Dafni Monastery-Comparison between surveyed cracks and regions of high tensile stresses (according to elastic analysis of the model of the structure).

In case of existing structures, the tensile strength of masonry may not be relevant because of the presence of cracks due to decay combined with previous mechanical effects.

Furthermore, the occurrence of cracks (meaning that at their location, the tensile strength of masonry is reached) does not necessarily mean that masonry is close to failure, as explained in more detail in the following Section 4.

Thus, normally, the tensile strength of masonry is not taken into account in assessment or re-design of masonry structures, with one exception though (see Section 5).

This is the reason why the verification of masonry structural elements is performed in terms of action-effects (forces, bending moments) and not in terms of principal stresses.

Nonetheless, preliminary analyses yielding values of principal tensile and compressive stresses (Figure 11) are very useful, as they allow the Engineer to identify

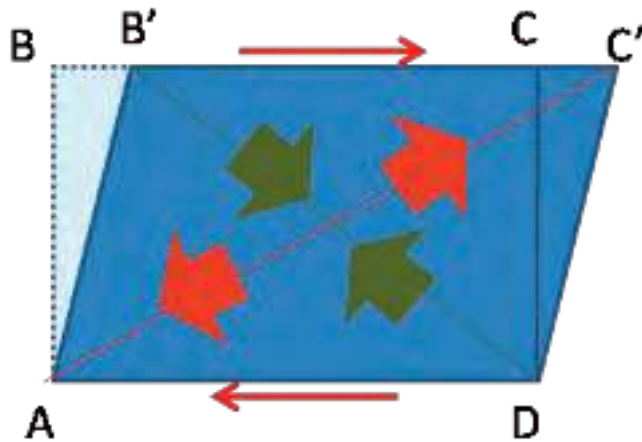
regions of high tensile stresses and, hence, to predict future or to interpret existing cracks in the structure.

4. Masonry under in-plane shear

This is one of the two (the other being the out-of-plane bending) major damaging causes for masonry structures subjected to seismic actions. Although well known, the mechanism of failure in in-plane shear is briefly presented here, as it gives the opportunity for discussion on several aspects that are, quite frequently, neglected.

A masonry element subjected to in-plane shear deformations (it is reminded that seismic actions do not induce forces, but deformations to the structure) is deformed, as schematically shown in Figure 12. One of the two diagonals is elongated (therefore, subjected to tension), the other being reduced in length (and, hence, under compression). Thus, the occurrence of an oblique crack (perpendicular to the elongated diagonal) is expected. The change of sign of the

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Figures 12a, 12b. Masonry elements under in-plane shear.

imposed shear deformations may or may not lead to the occurrence of an oblique crack perpendicular to the other diagonal of the masonry elements, depending on the amplitude of the imposed by the earthquake deformations (Figure 13).

In what follows, several aspects related to cracking and failure, as well as to the estimation of the bearing capacity in in-plane shear are presented and discussed upon:

(a) The morphology of shear cracks

Do shear cracks pass through mortar joints exclusively or do they cross masonry units as well? As shown in Figure 14, the morphology of shear cracks depends on several parameters, namely: (i) The relative mortar to masonry unit mechanical properties. Actually, in the usual for stone masonries case, where the strength of stones is by orders of magnitude higher than that of the mortar, shear cracks pass through mortar joints exclusively. On the contrary, in earth construction, where earth masonry units are of



similar strength with earth mortar, cracks are expected to cross both units and mortar. (ii) The size of masonry units and the thickness of mortar joints. As shown in Figure 14c, although before removing the plaster the crack seems to be rather like a straight line (insinuating that it crosses both mortar and masonry units), the removal of plaster reveals a masonry constituted by small size stones and thick mortar joints. Thus, the crack finds an easy way to travel through the mortar joints, without any need to deviate, in order to avoid the (stronger) stones.



Figures 13. Typical damage due to in-plane shear.



Figure 14. Morphology of shear cracks in 14a. Stone masonry, 14b. Earth masonry, and 14c. Masonry made with small size stones and thick mortar joints.

(b) How much close to failure is a masonry element in which shear crack(s) have occurred?

It is reminded that the shear strength of masonry is (following the Coulomb analogy) calculated as the sum of cohesion (c) and the friction along the horizontal portions of the crack (Figure 15). The contribution of

friction is calculated as the product of the friction coefficient (μ) and the vertical stress (σ). The occurrence of the crack, associated with measurable crack opening means that the cohesion is no longer present (it may be assumed to be reduced to zero). However, friction continues to contribute to the shear resistance of the element. Under some conditions, the resistance due to

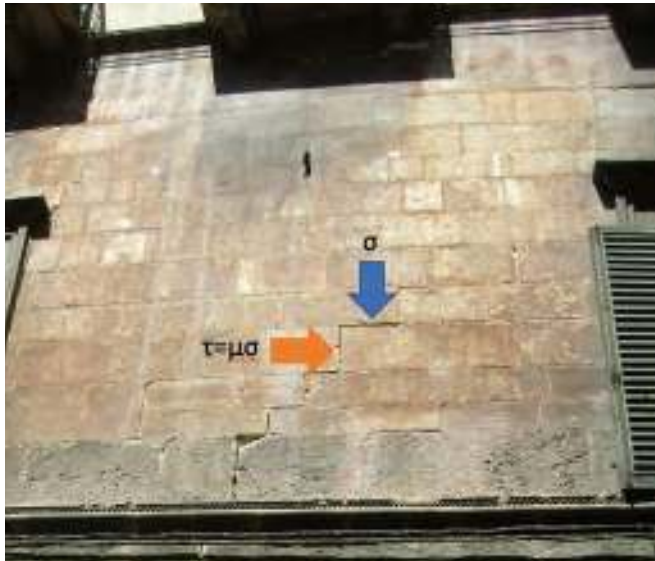


Figure 15. Shear transferred along cracks in masonry (friction mechanism).

shear may even increase after the occurrence of the crack, although the shear stiffness of the element may be reduced. Actually, under the assumption that further shear deformations are imposed to the element, the vertical portions of the crack open further (Figure 16). Thus, the horizontal length of contact between the two surfaces of the crack is reduced. However, as the vertical load remains more or less constant, the vertical stress on the remaining contact length of the crack is increasing.

On the other hand, the friction coefficient (μ) is not a material constant. It is well known that it decreases for increasing normal stress on the interface.

Thus, the product « $\mu\sigma$ » may-after the occurrence of crack-either increase or remain (for a certain range of imposed displacements) more or less constant or it



Figures 16. Walls subjected to large lateral displacements.



Figure 17. Failure of blocks in various locations along shear cracks.

may gradually be reduced. As a conclusion, one may say that the occurrence of shear cracks in a wall does not necessarily coincide with the attainment of the maximum resistance of the wall. Of course, under excessive lateral displacements, leading to large crack openings (as in case of the buildings shown in Figure 16), the contact of the two faces of the crack can be lost and the wall can be considered to be at its near collapse state. An alternative failure mode of elements subjected to large lateral displacements is that depicted in Figure 17: When the area of contact between the two faces of the crack is significantly reduced, the vertical stress corresponding to the reduced area may become very high and lead to the compressive failure of the masonry units. This failure mode is expected to occur more frequently to masonries made of rather weak masonry units, such as earth bricks.

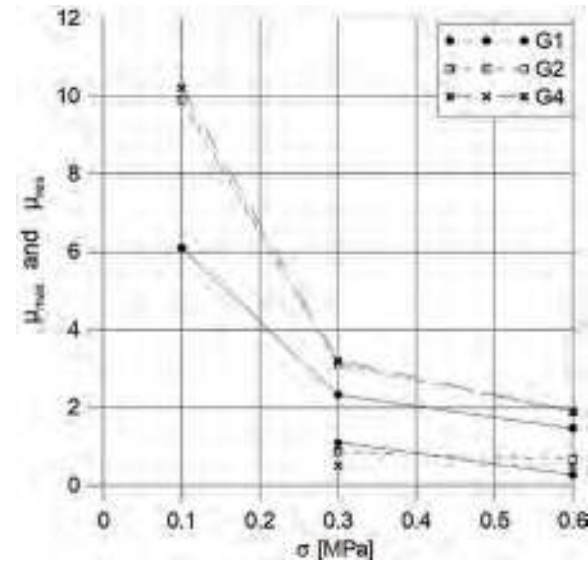


Figure 18. Friction coefficient as a function of the normal compressive stress [14].

(c) What is the adequate value of friction coefficient to be used for the calculation of the shear capacity of a masonry element?

The dependence of the friction coefficient (μ) on the value of the normal stress (σ) is well known. Tests on interfaces between stones or bricks and several grout mixes [14] have shown that for small σ -values (e.g. ~ 0.2 MPa), the friction coefficient can reach values as high as 4.0 or 6.0 (depending on the properties of the material of the joints, Figure 18). In any case, it seems that the value ($\mu=0.40$) adopted by the Codes (including EC6) for the calculation of the contribution of friction to the shear capacity of masonry is rather low, unless a significant vertical stress is imposed to the masonry element. Thus, when assessing the shear capacity of existing elements, belonging to a historical structure or a monument, it is advisable to

Source	Type of masonry	Measured values (MPa)	Remarks
Shahzada et al. [15]	Brick masonry wallettes	Very low	Not mentioned in the publication
Brignola et al. [16]	Stone masonry	0,040 to 0,067	In situ measurements (Tuscany, Italy)
Chiostrini et al. [17]	Stone masonry	0,061 to 0,160	In situ diagonal testing of wallettes
Corradi et al. [18] and [19]	Stone masonry	Similar to those measured by Chiostrini et al. [18]	
Vintzileou and Tassios [20]	Three-leaf masonry in diagonal compression	0,15	In-lab testing
Vintzileou and Miltiadou [21]	Three-leaf masonry in diagonal compression	0,10	In-lab testing
Milosevic et al. [22]	Rubble stone masonry	0,024 to 0,313	Irrespectively of the strength of mortar
Nikooravesh and Soltani [23]	Sun-dried bricks	0,01	Triplet test

Table I. Values of shear strength under zero normal stress (cohesion). Selected experimental results.

refer to adequate Literature and to estimate realistic values for the friction coefficient. Indeed, taking into account that conservative (and, perhaps, unrealistic) assumptions for the assessment lead to more heavy interventions (that may alter the Values of the CH asset), the Structural Engineer is invited to take profit of the advantage offered by the adoption of higher μ -values, if so needed and adequately justified.

(d) Are reliable values for the cohesion (c) available?

In-laboratory, the value of cohesion (or, in other words, the value of shear strength under zero normal stress) is measured by means of two alternative tests, namely: by imposing diagonal compression to wal-
lettes until they crack or by testing triplets (consisting

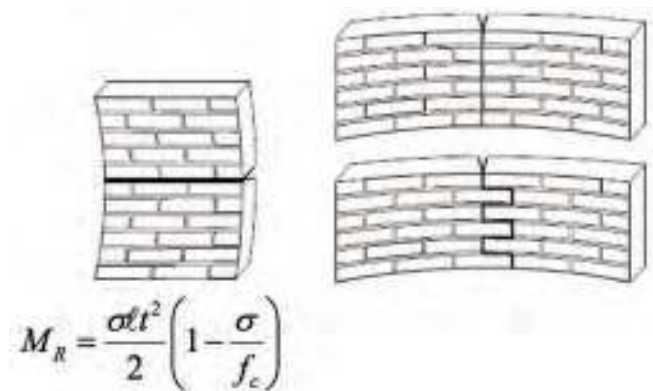


Figure 19. Out-of-plane bending of masonry; vertical or horizontal tensile stresses develop, depending on the aspect ratio of masonry.

in three masonry units and two mortar joints; the force is applied to the intermediate masonry unit). Unfortunately, the values of cohesion measured using either method are quite scattered, in case of historical



Figure 20. The out-of-plane bending of multi-leaf masonry elements may lead to separation among the leaves and to out-of-plane collapse of the outer leaf.

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masonries (either on site or in-laboratory on replicas). Some of the available experimental results (listed in Table 1) show the large scatter of cohesion, hardly attributable to any systematic cause.

Taking this into account, the Structural Engineer is advised to be quite cautious in selecting a value of this property for the assessment of a historical structure.

5. Masonry in out-of-plane bending

Codes for modern masonries (e.g. EC6) provide two values for the flexural strength of masonry, one for bending parallel to mortar joints and one for bending perpendicular to mortar joints. Those values represent actually the tensile strength of masonry subjected to flexure. The lower of the two values is valid for tension perpendicular to the mortar joints. However, the values provided by the Code do not take into account the beneficial effect of the vertical stresses (due to loads), which allow for higher bending moments to be resisted before masonry is horizontally cracked (Figure 19).

Thus, for the first case of Figure 19, it is suggested to apply (instead of the Code values) the formula allowing for the calculation of the bending moment the element can resist, taking into account the vertical compressive stress (σ) and assuming that, after cracking, failure is due to the attainment of the compressive strength of masonry at the compressed toe. In the formula of Figure 19, l and t denote the length and the thickness of the element and f_c denotes the compressive strength of masonry.

In the second case of Figure 19, on the contrary, as the vertical load does not contribute to the flexural capacity of the element, one has to take into account

a value for the tensile strength of masonry. This is perhaps the only case where the tensile strength of masonry is taken into account in verifications.

Obviously, the values provided by EC6 are valid for materials and masonry types used in modern masonry construction. They could, however, be used (with due caution) also for historical masonries, provided that the masonry elements consist in one single leaf or in more leaves solidly connected among them within the thickness of masonry. They may also be applied to masonry elements after the application of repair/strengthening techniques that ensure the connection of the leaves of masonry (e.g. grouting/homogenization of masonry or mechanical connection of the leaves using ties, etc.).

If this is not the case, due to pre-existing separation among the leaves or due to separation occurring due to the seismic event, each distinct leaf responds individually to the out-of-plane actions and mobilizes a resistance significantly lower than the expected for a solid element.

The most vulnerable leaf is, obviously the exterior one, since it is not provided with intermediate supports (on the contrary, the interior leaf may be laterally supported by transverse bearing walls or even by transverse partitions), whereas it may be not connected to the roof or to intermediate floors. In those cases, partial or full collapse of the exterior leaf may occur (Figure 20).

This is one more reason that makes the (as exhaustive as possible) documentation of the structure a prerequisite for its reliable assessment and, by way of consequence, for the selection of adequate intervention techniques.

6. Conclusions

On the basis of the material presented in this paper, the following conclusions can be drawn:

(a) Historic masonry is a general term, covering a vast variety of materials and construction types both on faces and within the thickness of elements. This variety is expected to be reflected also on the mechanical properties of historic masonries and on their behaviour, in general.

(b) The data that are available in the international Literature on the behaviour of historic masonries do not cover but limited construction types and combination of materials. Furthermore, behaviour models able to describe the behaviour of historic masonries are not available either, whereas modern Codes for masonry structures explicitly state that they do not cover existing masonry structures. It is to be noted that even Eurocode 8-Part 3, in its chapter on masonry structures provides very limited information related to the Mechanics of historic masonry.

Thus,

(c) Significant research effort is needed, at international level, for more and more reliable data to be acquired on the behaviour of historic masonry and masonry structures. In the meantime, as the built cultural heritage worldwide needs to be adequately preserved, the Structural Engineer has to base the assessment of historical structures and monuments to the available information, adequately and critically evaluated. To reach this, quite frequently, difficult goal,

(d) The Structural Engineer should, at a first step, document the structure as thoroughly as possible. To assist such documentation, there are several weapons ranging from visual inspection, to in situ

testing, to the application of non-destructive techniques (such as georadar, thermography, sonic techniques, etc.), to sampling and in-laboratory testing, or to laboratory testing of models at various scales (element, subassembly or entire structure). Another strong tool is

(e) The numerical modeling and analysis of the structure. Powerful software can provide significant information, under the obvious condition that it will be supplied with reliable data, resulting from documentation. Numerical models have to be adequately calibrated. One of the most valid ways to calibrate numerical models is to compare their results with the surveyed pathology of the structure under examination. Last but not least, the author of this paper wishes to stress on

(f) The value of engineering judgment, to be based on both adequate education and exhaustive documentation of the structure.

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Restoration and Strengthening Plan of the Bastam Minaret

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Abstract

The brick masonry minaret of the complex has been experiencing structural problems for many years. Due to uncertainty about its stability, it is necessary to evaluate it carefully and prepare its restoration and strengthening plan. The overall condition of the building containing the minaret seems good. The main concern about the stability of the minaret is the existence of a number of deep cracks and holes in the fabric of the minaret.

In this paper, activities carried out so far for the preparation of restoration and strengthening plan of the historical minaret of Bastam is explained.

Keywords: Bastam Minaret, Restoration, Petrography, Vibration Test, ISCARSAR Recommendations

1. History of the minaret and the historical complex

The city of Bastam is located at 6 km north-eastern Shahroud in the province of Semnan. It is connected to the Shavar mountains and the city of Mahan in the west, to the Miami region and Sheddad desert in the east, and to the province of Golestan in the north and north-west.

The historical complex of Bastam with many historical buildings and the mausoleum of the famous gnostic Bayazid Bastami is in the temporary World Heritage List. Because of the mausoleum of Imamzadeh Mohammad, the complex is visited by thousands of tourists every year.

The vast complex comprises the Seljuq and next eras heritage including the mausoleum and monastery of Bayazid, Safavid veranda, the minaret, the Bayazid mosque, the mosque of Bastam, Imamzadeh Mohammad mausoleum, Kashaneh tower, Ghazan Khan dome and Shahrokhieh madrass (Figures 1 and 2). The minaret was built on 1120 A.D.

2. General condition of the minaret

Figures 2 and 3 show the location of the minaret in the Bastam historical complex and the various parts of the minaret. Figure 4 shows the position of the minaret in the plan, as well as an elevation and an approximate section of the minaret. The general condition of the building in which the studied minaret is located seems to be in good condition (Figure 3a).

The main observed pathology that caused to investigate the monument, with the purpose of judging about



Figure 1. The position of the historical complex of Bastam on the 55°00'01" longitude and 36°29'03" latitude.



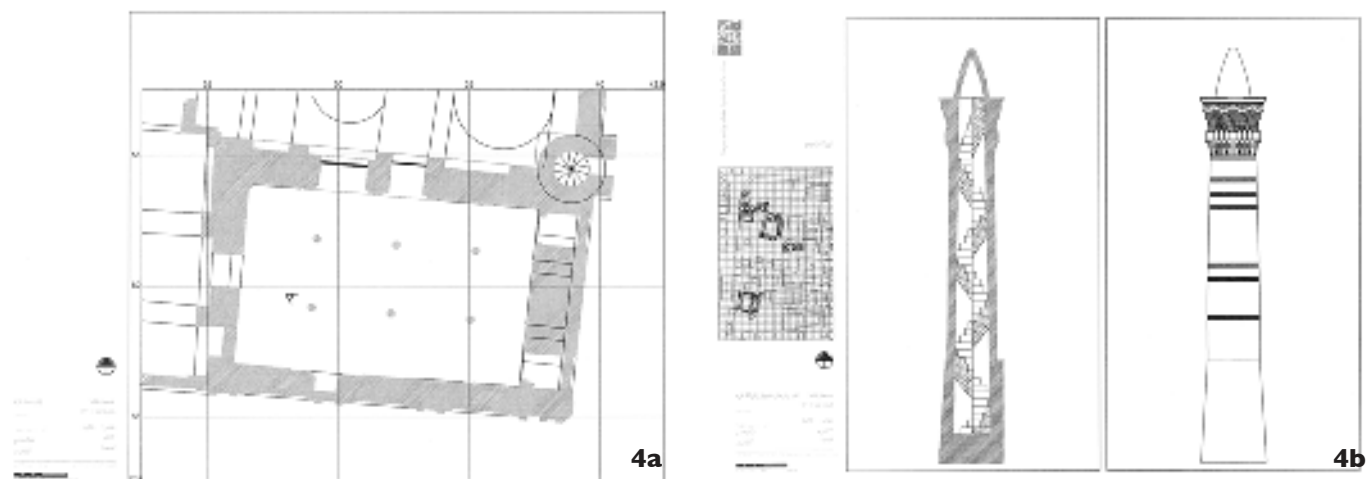
Figure 2. Historical complex of Bastam.



Figures 3. General photos and images showing different parts of the minaret: 3a. location within the complex, 3b. eastern side, 3c. eastern side, as photographed few decades ago, 3d. lower portion of western side, photographed few decades ago, 3e. entrance to the minaret (at its eastern side, 3f. lower side of the spiral staircase connected to the minaret shell, 3g. upper side of the spiral staircase.







Figures 4a, 4b. Location of the minaret in the plan of the complex; approximate section and elevation (from the archives of the ICHHTO in Bastam).



Figures 5. Recess at the external side of the minaret at two-thirds of its height: 5a. eastern side at present, 5b. eastern side, photographed few decades ago, 5c. western side at present, 5d. close-up view of the eastern side, 5e. recess at the eastern side (downwards view), 5f., 5g., 5h. recess at the eastern side inside the minaret (upwards view).



the stability of the minaret and the measures to be taken for its preservation is as follows: a) the recess of the northern side of the minaret at two-thirds of its height (Figure 5), and b) the cracks (visible on both exterior and interior face of the minaret shell) at different parts of the upper half of the minaret and a number of holes at mid-height (Figure 6).

3. Phases of restoration plan

The restoration plan of the Bastam minaret is on the basis of ISCARSAH recommendations [1]. The description of the phases of the restoration plan of the

Bastam minaret is presented in Table I.

Some of the phases are explained in the following sections, as the plan is not completed by the time of writing this paper.

4. Vibration test

In order to evaluate the dynamic properties of the minaret of the Bastam historical complex, a series of vibration tests were recently conducted (between 1 and 3 October 2018).

Taking into account the preliminary analyses of the minaret, adequately modified to account for the

Phase	Activity
1	Historical survey of the building
2	Archaeological survey of the building
3	Architectural survey of the building
4	Documentation of the present state of the minaret
5	Pathology of the minaret
6	Experimental tests for determining mechanical properties of original materials of the minaret+ Petrographic microscopic studies (thin section studies) including XRF and XRD tests
7	Experimental tests for determining mechanical properties of new materials used for restoration of the minaret+ Petrographic microscopic studies (thin section studies) including XRF and XRD tests
8	Excavation of pits to determine foundation condition and geology situation
9	Vibration tests
10	Structural analysis of original building to evaluate its seismic vulnerability
11	Selection of techniques and materials for restoration based on related international charters and laboratory test data
12	Design of structural strengthening and verification by structural analysis
13	Preparation of drawings and final report for restoration and strengthening

Table 1: The phases of the restoration plan of the Bastam minaret.

current state of the minaret, decisions were made about the location of the sensors (Figures 7a and 7b). For that purpose, a site inspection (foundation, interior of the minaret and roof of the main building) was also needed.

Taking into account: a) the condition of the minaret, and b) the available facilities, decisions were to be made on the means to apply for the excitation of the minaret.

Accordingly, to stimulate the initial modes, two sources of vibration: 1) vibration with a controlled shaking by hands at the highest point of the minaret (Figure 7(c)), and 2) the vibration from the wind blowing. To stimulate the higher modes, two other vibration sources

were used: 1) controlled impact using a rubber hammer, and 2) harmonic ground vibration using a heavy roller (Figure 7d).

Scaffolding was erected around the minaret a few decades ago and it was connected to the minaret at four points at four-fifth height.

In order to identify the effect of the scaffolding around the minaret on the vibration of the minaret, the operational modal test was performed in two cases.

In the first case, the scaffolding was connected to the minaret at four points at four-fifth height. In the second case, the scaffolding was disconnected from the minaret. The sensors were mounted at five different locations along the height on the minaret. Three



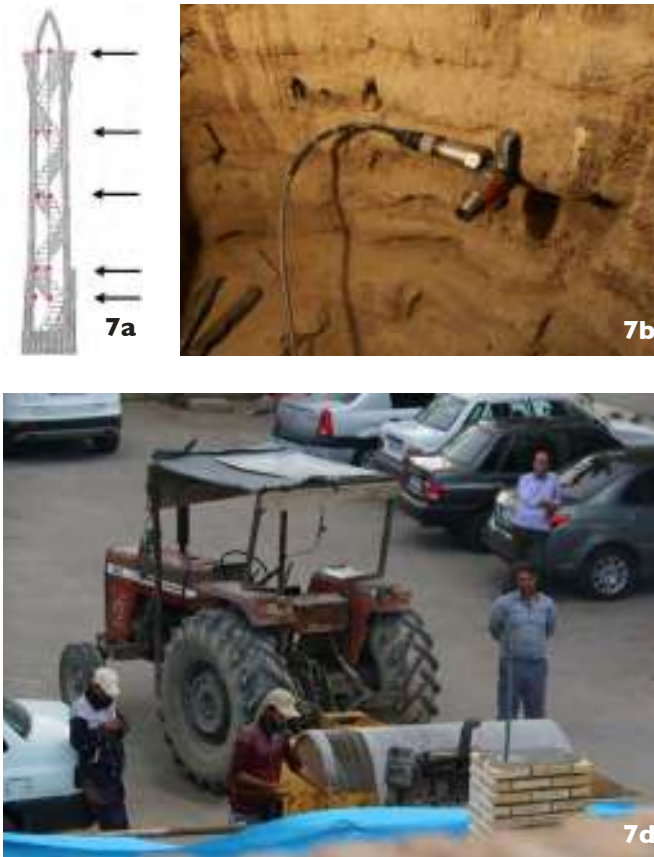
Figures 6a. Diagonal crack at the south-western side (filled with mortar), 6b. diagonal crack at the south-western side, photographed few decades ago, 6c. diagonal crack at the south-eastern side and vertical crack at the southern side, 6d. and 6e. external side of the hole at the northern side filled with mortar, 6f. internal side of the hole at the northern side filled with bricks, 6g. holes at the north-western side, 6h. and 6i. cracks inside the minaret, 6j. external layer on the main shell of the minaret near the apex.

experiments were needed for this purpose. Figure 7a shows the sensor installation operation. Figure 7b shows one of the sensors at position.

When the excitation was applied, the response of the structure was recorded through the sensors and stored. The graph of the power spectrum can be seen in Figure 8.

After the induced vibrations in the minaret were recorded, a vibration test was also carried out by installing the sensors on the body of main building on both sides of the minaret to record the effect of vibration on the building.





Figures 7. Vibration test: 7a. the locations of sensors (accelerometers), 7b. sensor at mid-height of the minaret, 7c. generating vibration in the minaret by hands, 7d. generating harmonic ground vibration using heavy rollers.

After primary data processing, removing noise frequencies and final data processing, natural frequencies of the minaret were determined (Tables 2). The determined main natural frequency of the minaret is 1.95 Hz.

5. Geotechnical studies

In a geotechnical study, the aim of the geotechnical study of the site was to determine the position and dimensions of the foundations of the minaret and adjacent walls, and to study the possible reasons for the deviation of the span.

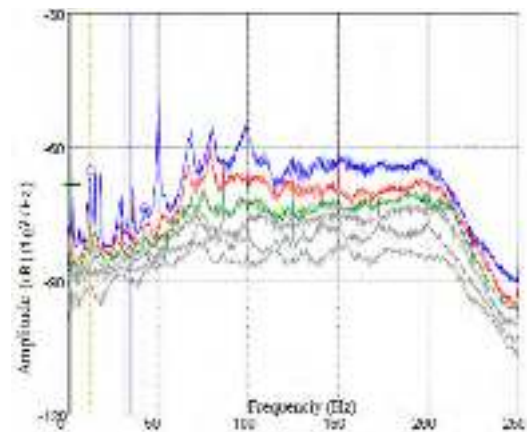
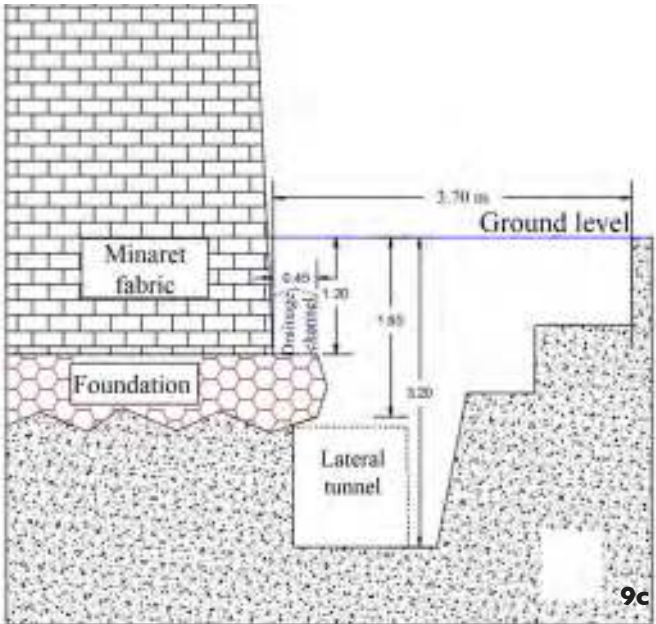


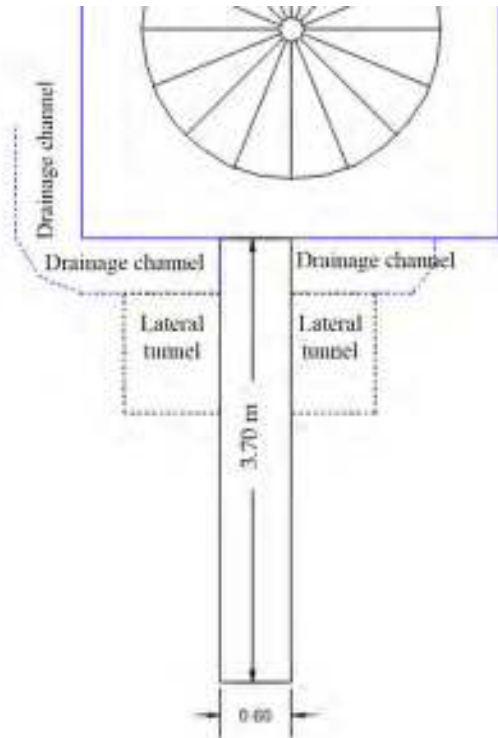
Figure 8. Power spectrum of recorded data.

Mode No.	Frequency (Hz)	Complexity (%)
1	1.953	0.502
2	12.451	2.323
3	30.518	27.95
4	34.912	17.693
5	42.236	59.941

Table 2. Frequencies determined from the frequency domain division method.



Figures 9a. At the eastern side of the southern wall adjacent to the minaret, 9b., 9c. and 9d. at the eastern side of the minaret.



9d

Based on this, in two points of the site two trenches were dug.

These two trenches with two already existing trenches were used for geotechnical assessment (Figure 9). The foundation is made of coarse to medium sized rubble of up to 40 cm in diameter, brick and lime mortar (Figure 10a).

Loose sand and silty soil with ash constitute the soil at the eastern side of the minaret (Figure 10b).

6. Mechanical properties of materials

Mechanical properties of the original brick, and original and new mortars of the minaret and materials of the minaret foundation have been determined through experimental tests.

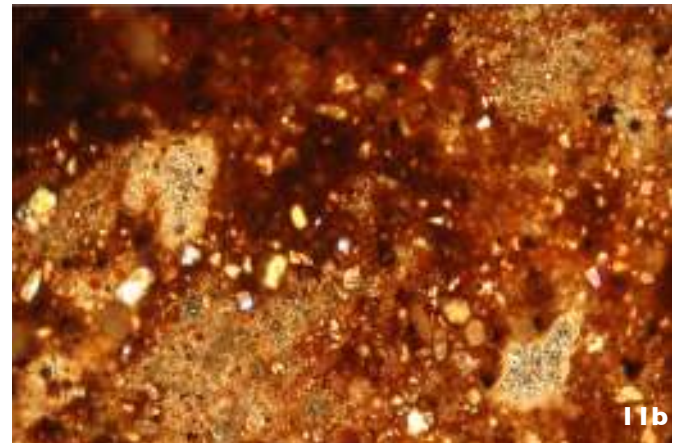


10a

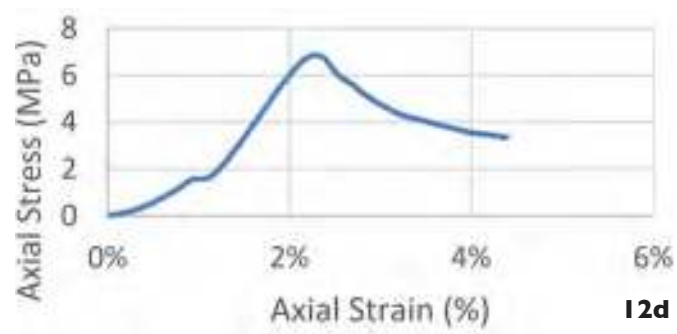


10b

Figures 10a. Foundation of the minaret protruded the upper wall (eastern side of the minaret), 10b. loose sand and silty soil with ash near the eastern side of the minaret.



Figures 11a. Lime crumbs in old brick of the minaret, 11b. lime and quartz crumbs in old mortar of the minaret.



Figures 12. Materials samples in the foundation: 12a. adobe, 12b. mortar, 12c. stone, 12d. stress-strain diagram of dry adobe.



Figures 13. Close range photogrammetry of the minaret, cloud point samples: 13a. outer side, 13b. close-up outer side, 13c. and 13d. inner side.

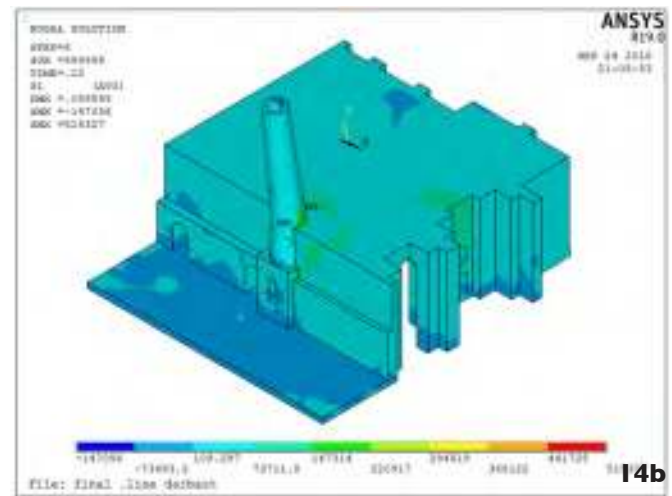
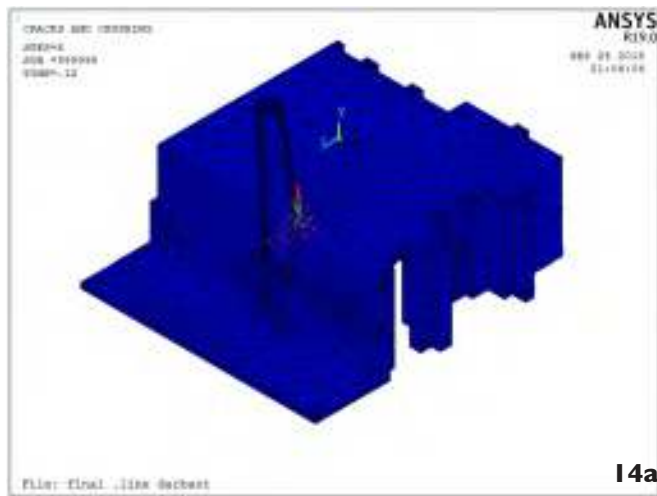


Figure 14. Initial updated finite element model of the minaret.

Stress-strain diagrams, modulus of elasticity and uniaxial compressive strength for dry and wet conditions have been determined (Figure 12).

7. Close range photogrammetry

In order to determine the conditions of the damages to the minaret and also its probable inclination a close range photogrammetry of the minaret was carried out. The preliminary result of the photogrammetry of the minaret is presented in Figure 13.

8. Structural analysis

Based on information obtained from above-mentions activities, the finite element model of the minaret and adjacent building have been made and calibrated (Figure 14).

9. Conclusions

Some phases of the restoration plan of the Bastam

minaret are explained on the bases of *ISCARSAH Recommendations*.

The phases of restoration and strengthening plan are the historical study of the minaret, archaeological study, architectural study, documentation of the existing condition, pathology, experimental tests for mechanical properties of the materials, study of existing materials (such as mortar and brick) for restoration (XRF and XRD tests), excavation of pits for examining the foundation and geology, vibration test, structural analysis for vulnerability evaluation, selection of techniques and materials for restoration on the basis of *ISCARSAH* recommendations and experimental tests, design of structural strengthening and verifications, and preparation of drawings and final report for structural restoration and strengthening.

The restoration plan is not completed yet and full explanation will be reported in near future.

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The First International Workshop on Restoration and Strengthening of Historical Structures

Iranian Ministry of Cultural Heritage, Tourism and Handicrafts (MCTH) and
International Scientific Committee on the Analysis and Restoration of
Structures of Architectural Heritage (ISCARSAH)

This set of two volumes present the outcomes of the *First International Workshop on Restoration and Strengthening of Historical Structures* (Tehran, Iran October 20th-31st, 2018) conceived as a forum for the exchange of knowledge, methodologies, and practical experience in the conservation of architectural heritage. Grounded in the principles promoted by ISCARSAH, the contributions emphasize an interdisciplinary approach integrating historical research, documentation, geometric survey, diagnostic techniques, structural and geotechnical analysis, material characterization, and conservation planning. International specialists examine the behaviour and pathology of historic masonry, methods of structural assessment, and intervention strategies aimed at reconciling safety requirements with respect for authenticity and cultural significance. Theoretical and methodological contributions are complemented by case studies and reports on numerous Iranian historic monuments and archaeological sites. Together, the papers and the reports demonstrate the importance of combining scientific investigation, engineering expertise, historical knowledge, and careful documentation in developing appropriate restoration and strengthening solutions. The volumes provide a methodological framework for professionals and researchers engaged in the conservation of historic structures, encouraging informed, compatible, and minimally invasive interventions while fostering international cooperation in the safeguarding of cultural heritage.

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