SHH07

STUDIES on HISTORICAL HERITAGE

Proceedings of the International Symposium

Antalya, Turkey
September 17-21, 2007

Edited by Görün ARUN

Organized by
YILDIZ TECHNICAL UNIVERSITY
RESEARCH CENTER for PRESERVATION of
HISTORICAL HERITAGE

TA-MİR

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PREFACE

Historical heritage that contain the architectural forms and the artistic values are under the danger of deterioration through the crucial environmental problems of urbanization and pollution, natural hazards, ignorance and new demands of the society. Safeguarding the continuously deteriorating material and construction of a historical complex, city or archaeological site with their natural and manmade environment necessitates integrated management and harmonious work of a multidisciplinary team of specialists dealing with history, urban planning, architecture, archaeology and different fields of engineering. Besides the interdisciplinary language the conscious participation of the society is also required so that the proper political decisions can be assured.

The International Symposium on “Studies on Historical Heritage” in Antalya, Turkey on September 17-21, 2007 is organized by Yıldız Technical University, Research Center for Preservation of Historical Heritage. The symposium is as a continuation of the previous international symposia entitled “Studies in Ancient Structures” held in Istanbul in 1997 and 2001.

The symposium provides an international and interdisciplinary forum for researchers, leading experts and people from application to exchange their experience and knowledge and disseminate information on preservation of historical heritage. Its aim is to enhance knowledge, increase awareness of the current technology and methodology and encourage studies of different disciplines working on historical heritage.

Studies on Historical Heritage highlight the state of the art in the diversity of professional skills, experience and knowledge necessary for preservation of historical heritage. Contributions of different disciplines from 22 countries contribute their own experience and attempt to express in interdisciplinary way the new concepts, technologies, methods and materials for the conservation and management of historic cities, sites and complexes.

These Proceedings containing papers sent by the specialists of different fields to the SHH07 Symposium is grouped according to their content, rather than according to their presentation, in order to make it more useful as a reference text. The Chapters are on: Historical Aspects and Documentation of Architectural Heritage and Their Environment; Archaeological Studies; Future of Historical Heritage- Heritage Management; Experimental Methods and Test Results of Materials; Structural Concepts**;** Intervention, Restoration And Preservation Techniques and Methodology. The author index is at the end.

The proceedings contain the papers that are reviewed. We wish to acknowledge and express our sincere gratitude to the Scientific Committee for spending their precious time in reviewing, editing and making significant recommendations to the authors. Special thanks to or keynote speakers; L. Binda, A. Galla, K. Kawaguchi, E. Madran, P. Roca, Y. Schaffer and T.P. Tassios whom we greatly appreciate their views on preservation of historical heritage. Of course, without the timeless efforts of the organizing committee members, this

symposium could not have been realized. Many thanks go to our sponsors and supporters for their invaluable and generous financial and technical contributions which indeed provide important link between the people in application and academia.

Finally warm thanks to all the authors from different parts of the world that have made considerable scientific contribution from their ongoing research activities. It is hoped that these contributions may be useful for professionals and researchers engaged in the problems of preservation and for those who have interest in the Studies on Historical Heritage

Dr. Görün Arun
On behalf of the SHH07
Organizing Committee
August, 2007

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CHAPTER V

Structural Concepts

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EVALUATION, EXPERIMENTAL METHODS AND TESTS

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ABSTRACT

Existing old structures or Monuments, are like voiceless black boxes; their geometrical, structural and physico-chemical characteristics are rarely known on available drawings and tests. Thus, structural evaluation and decision making regarding retrofitting or strengthening of a Monument, has to be based on historical information and, above all, on detailed measurements and experimental investigations. This lecture underlines the fundamental importance of these investigations, in order to minimize arbitrary assumptions and overconservative strengthening proposals.

1. INTRODUCTION

An evaluation of the bearing capacity of Monuments is needed for decision-making regarding both the technical methods to be used for repair or strengthening, and the optimisation of the consequences of the intervention on (i) the values of the Monument and (ii) on the related Social values. Such evaluation, be it qualitative or quantitative, is based on several sets of historical, geometrical, physical-chemical and structural information. This paper is an attempt to enumerate and very briefly comment upon the experimental measuring and testing methods offering such structural information. In doing so, however, preference will be given to the more modern methods rather than those better known to our profession.

2. INVENTORY OF SCOPES AND CORRESPONDING EXPERIMENTAL MEANS

It is worth to inventorise first the categories of structurally useful data needed (thus making clear the scope of the experimental investigation), together with the particular methods used to this end.

2.1. Instrumental in-situ methods

a) Geometrical data

- Visual description of structural parts: Laser scanning, Photogrammetry
- Cracks
  - Opening: Lenses, Photogrammetry, Strain gauges
  - Depth: Ultrasonic tests
  - Length: Measuring tape, Photogrammetry
- Displacements: Photogrammetry, Inclinometers, Penduli

b) Internal structure, [1-6]

- Hidden voids or discontinuities: Endoscopy, Thermography, Radar, Sonic tomography, Radiography
- Internal building details: Endoscopy (see also BIPS-method in § 5), Radar, Sonic tomography
- Hidden metals: Magnetometry, Radiography, Thermography, Radar

c) In-situ strength of constitutive materials

- Stones: Rebound, Ultrasonic, Scratch width
- Infill material: Sonic cross-hole
- Mortars: Scratch width, Penetration test [7]
- Timber: Penetrometer [8]
- Metals: Hardness test in-situ
- Bonding strength: Mortar pullout test

d) Structural properties of masonry

- Acting compressive stresses
- Compression resistance
- Shear resistance

In situ jack-tests [3]

e) Effectiveness of Grouting: Sonic tomography, Endoscopy, Radar [9, 10]

f) Dynamic response of building elements

- Microtremors
- Cable-release tests
- Vibrodyne

g) Subterranean data

- Seismic tomography
- Ground radar

2.2. Laboratory methods

a) Core testing: Compression, Tension
b) Tests of irregular mortar-fragments
c) Testing on replicas: re-made masonry, subassemblages

2.3. Monitoring (in-time)

a) Displacements: Horizontal deformetric wires, Penduli, Laser measurements, Inclinometers
b) Settlements: Leveling systems, Inclinometers
c) Internal forces: Inserted dynamometers, Flat jacks
d) Humidity within masonry: Neutron probes, GPR, Thermography
e) Ground - water level: Water pressure borings
f) Cracks: Opening evolution and control
g) Seismic actions: Seismometers, Accelerometers
h) Environmental data: Temperature, Solar radiation, Wind

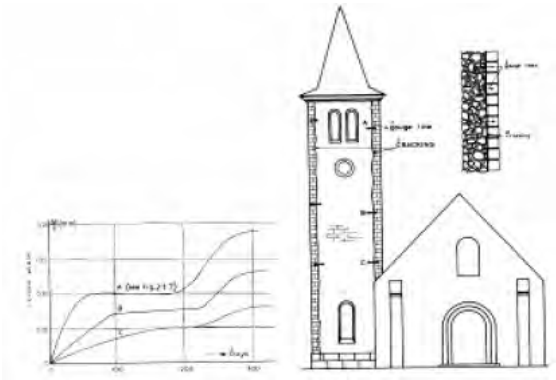

Figure 1: Vertical crack widths on this steeple (France) were followed up for one year [11]

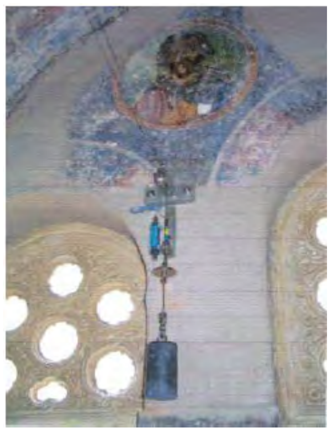

Figure 2: Displacement transducer, installed in the Katholikon of Hosios Loukas Monastery, Greece (photo A. Miltiadou)

For our purposes, it is good to know that such a rather impressive weaponry is made available to us, in order to "see" the interior of the (black and silent) "box" of the structural elements of a Monument.

A lot of the aforementioned methods are well known to the Engineers; several books and papers describe them in detail (see i.a. [11]). Consequently, only some of these methods will be briefly described and commented upon in this paper, together with some applications.

3. GEOMETRICAL AND STRUCTURAL DATA

In order to appreciate the use of Experimental Mehods (E.M.'s) in evaluating the structural conditions of Monuments, a "problem-solving" presentation is followed here: for each of the problems to be solved, the role of appropriate E.M.'s is mentioned, along with a minimum of comments.

3.1. General Layout

A combination of historical, visual and photogrammetric data is normally used. All geometrical data are found.

3.2. Identification of the Structural System

a) Connection of transversal walls: Radar, Endoscopy, Sonic cross-hole
b) Hidden openings and in-plane discontinuities: Thermography
c) Discontinuities and voids across the wall: Radar, Endoscopy, Sonic cross-hole
d) Dynamic tests: Identification of natural periods and vibration modes

3.3.Resistances of critical regions

a) Compression resistance of masonry

- In-situ flat jack tests
- Use of realistic prediction formulae, with the following input data
  – geometry: as in § 3.1 and § 3.2c, including the thickness of mortar joints
  – mortar strength: fragments' testing, scratch width, mortar penetrometer
  – blocks' strength: ultrasonic, scratch width, rebound test
- Testing on full scale replicas (in-lab)

b) Tensile resistance of masonry along the principal directions x,y:

- Roughly, as an appropriate (and variable) percentage of compression resistance
- Analytically, based on:
  - friction tests along joints
  - tensile strengths of mortar and blocks
- Experimentally, on replicas
  - tests along the principal directions
  - "diagonal compression tests": due corrections should be made to account for oblique rupture of blocks and for peak σ_t/σ_c ratios.

c) Shear resistance of masonry

- versus sliding failure: as per § 3.3a, together with friction tests along mortar joints
- versus "diagonal" failure: it is recommended to use analytical methods (applied for given σ_t/σ_c ratios) and making use of the aforementioned mechanical data.

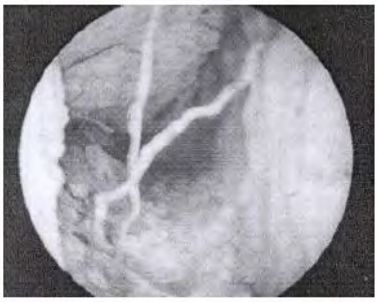

Figure 3: Endoscopy through a masonry wall of Dafni monastery, Greece. Mortar between stones is disintegrated; roots of a plant can also be seen [12]

Figure 4: Endoscopy: A sound timber element is disclosed within the wall. Panagia Krina in Chios Island Greece (photo E. Vintzileou)

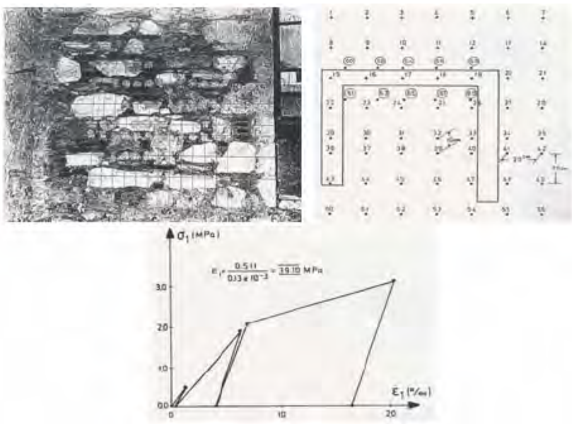

Figure 5: In situ compressive strength evaluation of low quality stone masonry, by means of flat-jacks [13]

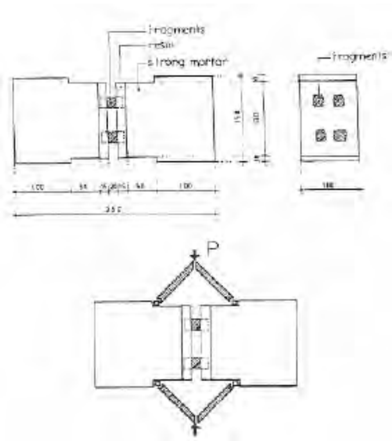

Figure 6: Mortar-fragments' test: Determination of tensile strength [14]

4. EVALUATION OF THE BEARING CAPACITY OF THE MONUMENTS

4.1. Analysis and Calibration

a) Normally, a step-by-step seismic loading is applied, so that any damage produced by a previous step of loading will be taken into account as a modification of the rigidity matrix used for the subsequent step of loading.

b) In any case, the best way to confirm the suitability of the analytical tools used is their capacity to "predict" analytically the damages of the Monument under specific conditions: the results of a previous earthquake may therefore be a good opportunity for us to "test" the validity of the analytical methods to be used. Such a calibration of analytical methods seems to be a sine qua non condition for their reliability.

4.2. The concept of acceptable damage level

a) In general terms, a certain ground acceleration may be considered as "critical" if it was found to produce a limit damage level, beyond which any further damage would be unacceptable. Such "just acceptable" limit damage-levels could be described as follows: (i) Slight local fissures at the corners of some openings of masonry walls or very small local slidings of large blocks or column drums (in case of Greco-roman antiquities) (ii) Systematic cracking of walls, or considerable displacement of large blocks or column drums (iii) As § ii, plus local compression ruptures of building elements etc. (iv) Partial collapse of some building elements, without however any global collapse.

Obviously, higher damage-levels are accompanied with higher energy dissipation and, consequently, with higher nominal seismic bearing capacity of the given Monument.

b) The concept of acceptable damage-level, however, can only be practicable if additional classifications of Monuments are available, regarding Importance-level and Visitability-level. If this is so, acceptable damage-levels might be assigned, the way it is indicatively shown in Table 1. Obviously, such classifications should be decided by the appropriate State Cultural Authorities.

Table 1: Indicative acceptable seismic damage-levels I, II, III, IV of Monuments

		Regarding Protection of human life			Regarding History and Aesthetics
Visitability		V1	V2	V3	
Importance of the Monument	I1	I	II	II	I
	I2	II	II	III	II
	I3	III	III	IV	III

5. INDICATIVE APPLICATIONS

This lecture could not be a Manual of available E.M.'s in structural investigations of Monuments. Selectively, however, a series of practical applications are illustrated in what follows; short captions offer the necessary explanations.

Besides, Table 2 summarises the main characteristics of the most modern investigation methods. The significance of combined Experimental Methods of investigation is also stressed out: such a redundancy of information may balance the limitations of individual methods; whenever possible, full replicas of wallets may be constructed in Laboratory and tested.

A relatively recent development in Endoscopy should also be mentioned here, the so called "Borehole Image Processing System" (BIPS): It allows to see in colour the entire cylindrical surface of the boring, developed on the screen in real time (e.g. see Fig.7). And this seems to be an extraordinary rich instrument for Structural Engineering.

Figure 7: Borehole Image Processing System (BIPS). The entire cylindrical surface of the boring is developed and examined in the case of an endoscopy in the Tower of Pisa, near the deficient area of the helicoidal stairs [15]

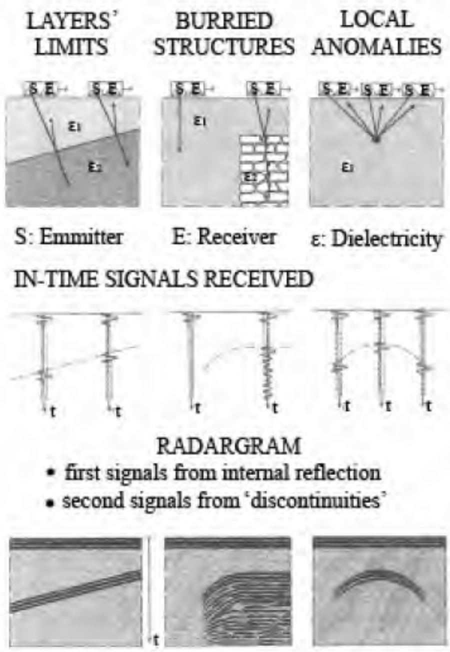

Figure 8: Principle of Ground-Penetrating Radar (GPR) [16]

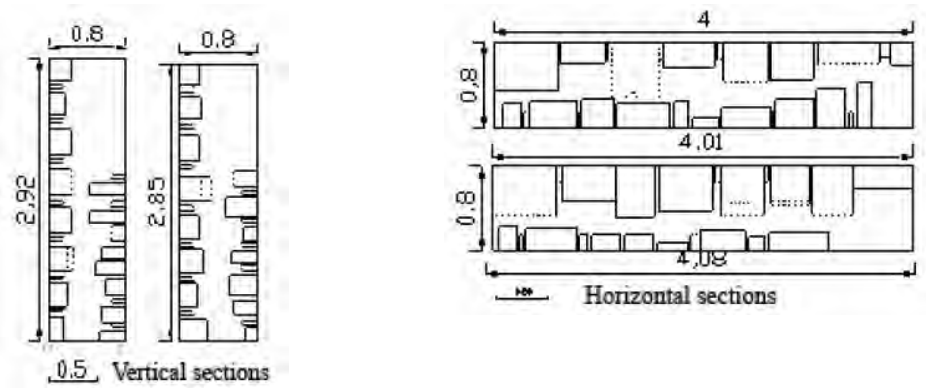

Figure 9: Thanks to a combination of Radar and Endoscopy, an almost complete image of the internal structure of the upper parts of walls was acquired. Katholikon of Dafni Monastery, 11_{th} century, Greece [12]

Table 2: Main characteristics of modern ND methods

N^o	Method	Waves	Frequency (approx)	Range	Captors
1	Sonic (seismic)	Mechanical (pressure transmission waves)	0,3 - 5,0 KHz	>>	On opposite sides
Technical adverse condition: Water saturation; in media composed of parts with different moduli of elasticity, refraction of waves produces a curvilinear transmission ("wave delay"); intensive traffic (in case of ground seismic investigation)					
2	Ultrasonic	Mechanical (pressure transmission waves)	20 – 100 KHz	<<1 m	On opposite sides or both on one face
Technical adverse condition: Surface roughness; strong non-homogeneity					
3	Radar (GPR)	Electro-magnetic (reflections waves)	50-1500 MHz	Wall <1,0 m Ground <2,0 m	Both on one face
Technical adverse condition: metals near face; false floors; clay plate covers; non-uniform humidity (dielectricity increases by ~150% for 10% humidity content; this allows the use of GPR also for systematic humidity investigations, within a relatively uniform medium)					

Sonic (seismic) methods, however, remain the most powerful global investigation methods. Relatively low frequency mechanical waves are produced by percussion on one side of the medium under investigation, whereas on the other side, appropriate receivers offer the possibility to measure time, thus to evaluate velocities across the given medium (a ground area or a wall). Multiple captors' arrangements and computer analysis allow for a sonic tomography to be produced, thus mapping areas with different velocities, i.e. with different structural properties. In the case of masonry walls, "cross-hole" sonic methods may offer much more precise information along the wall, such as the assessment of the thickness and the quality of the infill in case of three-leaf masonry!

Finally, it is interesting to note here the usefulness of permanent monitoring as a means to improve our knowledge of the real structural system: In the case of the Brunelleschi dome (Florence), it was proved that "the dome does not behave like a dome (i.e. with radial symmetry), but as four almost independent sub-structures, separated by the main (vertical) cracks" [17].

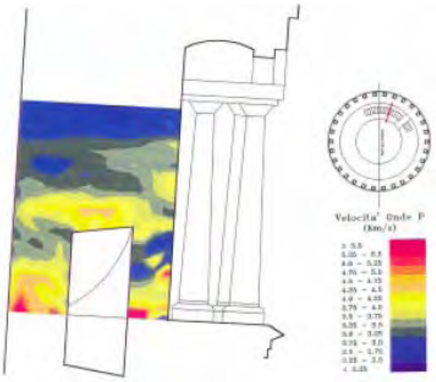

Figure 10: Sonic tomography of horizontal section, in a deficient area of the Tower of Pisa [18]

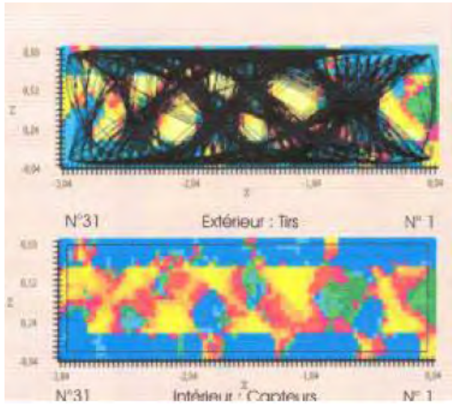

Figure 11: Sonic tomography along a wall of Dafni Monastery, Greece, After grouting [19]

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STRATEGIES PREVENTING WEATHER DAMAGE ON ARCHITECTURAL HERITAGE

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ABSTRACT

The paper deals with study of damages on architectural heritage caused by weather effects, with collection of data on damages and failures, discusses special cases of their experimental modelling, their threat to different historic structures and elements and presents preventive measures and strategies against the weathering effects. It presents selected examples from a developed information system containing data on typical damage and failures related to weathering effects and disasters action, namely wind, floods and landslides.

1. INTRODUCTION

Weather effects represent one of the most deteriorating agents to cultural heritage, namely architectural heritage. The sensitivity of historic structures and elements to weather and disasters has been studied using records of damage and failures gathered in literature and special databases. It should be emphasized that any historic structure or element cannot be dealt with unless its material characteristics and a general environmental situation, (e.g. hydrogeological conditions, air pollution, …), have been taken into account, which is reflected in the ranking results and mitigation strategies.

Analysis of failures of cultural heritage buildings and design of appropriate remedial or mitigation measures need a systematic study of damage mechanisms, damage manifestation and failure patterns. This can be effectively done only with the support of relevant and up-to-date databases, information systems and in some cases also special investigations utilizing methods of experimental structural and material research. The limited extent of the paper cannot deal with this problem in detail, and only a suitable methodology is shortly discussed.

2. TYPICAL DAMAGES AND FAILURES ON HISTORIC OBJECTS

The lifespan and serviceability of materials, elements, buildings and ensembles of architectural heritage are influenced by various factors, including to which type of action they are subjected. A great deal of attention is devoted to those which affect the physical existence of the object or building. Nevertheless, almost all intangible heritage features and values, especially historic cities, can be severely damaged, which is definitely a defect and can even attain the dimension of a failure. We can divide the aforementioned factors causing the deterioration of the physical state of monuments into three categories according to the period of their action: i) before the production of the object or building, ii) during the production/construction phase and iii) during the service life of the object or building, (Drdácký 2001). Even though historic buildings exhibit numerous defects and failures caused by factors originating before their construction, which is an important issue for conservation theories and approach definitions, (Drdácký, Slížková 2006), the third aforementioned category is the most important one for architectural heritage, and it can be divided into the following groups: weathering effects, biological factors, incompatibility, loads, operation, concept, design, materialization and alterations to the building or object, unexpected events & “injuries”. Climate change influences or acts in synergy with all the aforementioned factors which are summarised as follows. Weathering effects involve namely action of radiation, heat, water, air, wind and air pollutants. They affect cultural heritage mostly in a synergic way and occur in both the exterior and the interior spaces.

2.1. Damage and failures

The arrangement and classification of damage and failures can exploit different points of view, because of their vast variety. Let us briefly mention some of the classification approaches. Failures are usually sorted according to their *consequence* (defects, relevant damages, failures), according to materials or structures made from these materials. In the architectural heritage field we distinguish between failures of stone, stabilised clay, burnt clay, mortars and plasters, concrete, wood, metals and plastics. Another possibility takes advantage of building typology, e.g. houses, buildings, halls, towers, tanks, bridges, roads and railways, sculptural works, etc. In buildings we frequently use the classification of structural typology or structural function, sometimes with the incorrect terms – bearing and non-bearing structures, (which neglects the fact that any structure has been created to sustain some loads or action, therefore as the “bearing” one). In this contribution, we accepted the sorting of damage and failures according to their manifestation, their relation to the selected causes of damage and failures and to possible preventive measures which are dependent on structural typology and especially on the material used.

All abovementioned criteria are very important characteristics of damage and failures, therefore, they should be recorded in the relevant databases. There might be terminological problems because there is no widely accepted system for classifying damage of monuments. In such a case it is useful to use terminology developed and introduced for other purposes, e.g. listing of cultural heritage.

A very detailed system for material damage diagnosis on stone monuments has been developed by Fitzner and Heinrichs 2002, who classify weathering forms into four levels with four groups of weathering forms. Similarly, a more practical system has been suggested by Smith, Whalley and Magee 1992. This paper is rather focused on structural and architectural features.

2.2. Damage mechanism

Even though this paper is not primarily focused on material damage assessment, it is useful to remember that "sound" material is a necessary condition for physical existence and sustainability of architectural cultural heritage. Knowledge of basic damaging mechanisms of historic materials is indispensable for their appropriate and effective protection and safeguarding. In principle, the historic materials are deteriorated by means of three mechanisms which in many cases act simultaneously or which at least interact together.

Physical damaging

Material deteriorates mainly mechanically due to the action of external forces (from loads, movement, impacts, human action, etc.) or internal forces, (e.g. generated by forced deformations at uneven temperature and moisture changes). The time factor may be important for some materials, e.g. long term overloading of timber. Further, erosion problems decreasing cross-sectional characteristics belong to this group, too. Physical damage typically results in a mechanical breakdown, for stone typical examples see in Smith et al. 1992.

Chemical damaging

The material is chemically attacked by reactive compounds present in the surrounding environment or produced by biological agents. In the second case we distinguish between assimilation and dissimilation damaging. In the former case, the material serves as a nutrition source for parasite organisms that e.g. decompose cellulose in organic materials. In the latter case, products released from colonised organisms without nutrition aims cause deterioration. Chemical damaging can be initiated or accelerated by means of physical factors, e.g. temperature or light, (or another radiation), and then we speak about thermal damaging (well known for timber and marble) or photochemical damaging.

Soiling

Soiling is a general phenomenon which decreases the serviceability of historic structures and elements. It may generate only aesthetical defects but also more serious failures, e.g. reduction of pipe profiles, bridging of electrical circuits by mycelium, creation of a dark crust causing further increased surface heating and elevated temperature stresses. The material is here frequently only a carrier of the undesired layer which might initiate or accelerate the above mentioned degradation types. Soiling can be detected by alterations, deposition and biological colonisation.

2.3 Typical damage categories and manifestation

When classifying damage and failures we usually distinguish among:

- No damage or failure
- Destruction, total collapse or failure of a part of an object/building
- Relative displacement or rotation of a part of the whole object
- Deformations – elastic or irreversible (deflection, buckling, compression, …)
- Cracks in structures (including detachment and delaminations)
- Loss of material (corrosion, woodworm, …)
- Material disintegration (life time exhausted, alteration, ...)
- Moisture (capillary, leakage, condensation, hygroscopic, ...)
- Biological attack
- Material or structural heterogeneity
- Appearance changes (efflorescence, discoloration, structural change, soiling)
- Alterations (porosity change, salts, wrong changes, …)

The above-mentioned categories are very rough and in some cases overlapping. For practical use they are more structured and refined, and completed with typical samples etc. As it has been already mentioned, the most elaborate approaches are known from the field of stone damage assessment (Fitzner, Heinrichs 2002, Smith et al. 1992) and corrosion of metals. The stone deterioration is very closely related to climate effects, and therefore, the ranking of structures and elements reflects this fact.

Determination of causes of damage and failures is the main aim of the failure analysis. On the basis of failure analysis preventive measures can be developed and applied. From such an analysis we know that most failures have not only one cause but they are a result of combinations of unfavourable conditions. Only about one third of failures might be attributed to one cause only. The other two thirds have statistically on average about 2.5 reasons, but in some cases failures have 7-8 causes.

Selected cases of damage, presented here, are only illustrative examples which do not involve all possible causes, situations, materials and structures.

3. RANKING OF STRUCTRES VULNERABLE TO WEATHERING

Taking into account the sensitivity of structures and elements to weather effects regardless materials from which they are created, we logically sort the structures according to their exposure conditions and morphological characteristics which influence water retention or the eddies and flows around the monuments. The last item is reflected in the robustness categories in which material types are also mentioned.

3.1 Objects exposed to rain

Sculpture and building surfaces ***exposed to rain*** are divided into five categories: R0 Sheltered from rain; R1 Partly exposed to rain or/and moderate rain water runoff (vertical surfaces moderately exposed to winds); R2 Exposed to rain or/and heavy rain water runoff (roofs, inclined surfaces of sculptures, vertical surfaces exposed to prevailing and strong winds); R3 Complex shapes with horizontal surfaces (cornices, balconies, decorative architectural elements); R4 Complex shapes with water traps (roof and façade details).

3.2 Objects threaten by rising dampness

Structures and elements exposed to threat of ***rising dampness*** are classified as: D0 Natural moisture contents ; D1 Occasional moisture occurrence (walls which are damp due to splattered water, e.g. bridge parapet walls, structures occasionally affected by an elevated water table); D2 Permanent moisture contents (masonry highly contamined with hygroscopic salts or materials, masonry covered with damproof layers); D3 Permanent high moisture contents (buildings in lagoons, cellars or caves); D4 Cyclical alterations under water/open air (bridge piers, watermills, marine structures).

3.3 Objects exposed to sunshine

Structures and elements ***exposed to sunshine*** (heat and light effects): S0 Sheltered against light; S1 Sheltered against direct sunshine; S2 Partly exposed to sunshine (buildings in shadow of trees); S3 Fully exposed to sunshine (south-east oriented façade); S4 Fully exposed to sunshine with elevated heat exchange (south + SW +W oriented facades or roofs, flat roofs and pavement).

3.4 Objects sensitive to robustness

Robustness (***volume of mass, detailing and material***) of the element or structure: M0 Robustly resistant to weather effects; M1 Robust in form, sensitive material (porous materials, materials with low freeze/thaw resistance, water soluble materials, corrosion prone materials - alterations, crust creation); M2 Resistant material, sensitive details (details with notches - stress concentration, manufacturing defects); M3 Form prone to uneven strains and gradients (edges, corners, protuberances, subtle elements fixed to massive parts); M4 Sensitive material & sensitive details.

4. STRATEGIES & MEASURES MITIGATING WEATHER EFFECTS

4.1 Regular inspection of structural health

Regular inspection of structural health concerns all categories of vulnerability to the weathering effects and represents a basic condition for properly and in-time maintenance of historical objects and prevention of damages and failures.

The scope and procedure of the regular inspection is generally oriented to checking of historical materials and structures irrespectively to a probable deteriorating or damaging action, (mainly for economy reasons). There are several guidelines available for such an inspection, e.g. The ISE guidelines.

From the weathering point of view the surface layers and elevated moisture are of the great importance. Therefore, the tightness of the envelope against penetrating water or snow, (at reasonable diffusion characteristics), and the damp rise structures and voids must be carefully checked.

4.2 Long term monitoring of structural health

Selected objects of higher cultural historical value or objects made form weathering sensitive materials (namely some stones) or fragile structures are subjects for long term monitoring of structural health. Optical devices and special sensors are mostly used.

4.3 Installation of warning systems

Regular monitoring and checking of roof lofts, timber and other moisture sensitive materials can be advantageously enhanced with evaluation and assessment systems which can serve as early warning indicators.

4.4 Regular maintenance

Regular maintenance significantly improves life cycles of historical materials and objects. It prevents development of serious damages from weathering action and prolongs time between restoration works. Here cleaning and renewal of protective paints or penetration represents one of the most desirable measures. “Soft” techniques must be applied and only the deposits which prevent “breathing” of object skin are to be removed. All facilities for water drainage must be regularly cleaned and kept in function.

4.5 Water disposition

Water in all forms is considered the most harmful agent for historic materials and structures and the water carry away must be fast and effective. All parts where water can accumulate must be provided with adequate water outlets. The expected increase of precipitation amount calls for appropriate changes in dimension of gutters. Water must be carried away from the vicinity of a building, not only from the roof and then left to moisture the subsoil around foundations.

4.6 Surface protection against water penetration

Prevention of water penetration or soaking into material is one of measures which are considered indispensable in some cases, as e.g. for ceramic roof tiles. On the other hand, an opinion of professionals on their use for façades is not so uniform. There exists a wide scope of materials for surface protection – from invisible agents to paints, namely the so called sacrificed paints or layers. Even though in the majority of European countries the hydrophobization of stone façades is not accepted, for stone sculptures it seems to be quite common.

4.7 Protection against excessive heat & light variations

Historic materials and structures can be effectively protected against excessive heat and light action by means of different shelters, filters and coats. In rooms, curtains and window shutters can be installed.

4.8 Architectural improvements

A specific problem arises in the restoration of monuments with historical (authentic) faults. During restoration works the faulty cornice details are usually restored with original errors. This approach should be changed, and the durability should be superior to formal copying of original details or repairs of appearance in justified cases. This applies primarily to the remediation of natural expansion cracks made without an adequate technical solution.

Typical problems involve: missing expansion joints (unaccommodated volumetric temperature and moisture changes), faulty details (mostly cornices), unsuitable materials, insufficient protection against climatic factors, rising damp. The faulty details should be improved in order to prevent adverse consequences.

4.9 Replacement of originals by replicas

Severely deteriorated or very precious heritage objects subjected to harmful effects can be saved by replacement of the original with a replica. Such a method is quite expensive but relatively safe if the original is placed into environment ensuring physical and chemical conditions necessary for its long term sustainability.

5. EXPERIMENTAL RESEARCH

The study of weather effects on architectural heritage requires long time monitoring of real situations and theoretical as well as physical modelling. In the experimental field, the research has been mostly focused on material testing in climate chambers, and practically no attention has been paid to physical modelling of real full scale or reduced scale situations, except of studies on the air flows around buildings or over territories. Possibilities of physical modelling focused on complex weather phenomena have been suggested and tested.

For example, soiling and deposition effects were studied on the Prague National Museum in reality as well as on two reduced scale models (1:100 and 1:50) which were tested in a French CSTB climate tunnel “Jules Verne” in Nantes. Another methodology concerns full scale testing of the combined effects of rain, wind and temperature cycling on porous materials, which has been tested in the CSTB climate tunnel, too. In this case full scale architectural elements have been tested in two options – without any surface treatment and with a water repellent surface coating.

Further, the measurement of wind forces on historic buildings differs from the common test on modern structures. The developed methodology brings together wind tunnel measurements made in Prague BLWT (Boundary Layer Wind Tunnel) with full scale measurements, both historical measurements and measurements made during the course of a study, and CFD calculations. This results in a unique data-set which provides a basic understanding of the actual mechanism of drift built up in combination with strong and moderate winds. Several typical shapes for the historic roofs and structures have been selected, mostly for old town towers. Though variable in many details, they have been diversified into main two groups according to the aerodynamic shape of particular details of interest (i.e. pinnacles, cupolas, etc.) that very often characterise two major architectural styles; Gothic and Baroque. Computer models of the flow around those typical tower structures including particular and most exposed details have been developed in two- and three-dimensional space.

6. CONCLUSIONS

Weather effects are going to be intensified with the contemporary apparent tendency of climate change and they need to be considered as a natural hazard with long time action and disastrous consequences on sustainability of cultural heritage. Appropriate measures are to be developed and applied at various types of cultural heritage objects ranked according to their vulnerability.

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SEISMIC VULNERABILITY ASSESSMENT OF HISTORICAL CONSTRUCTIONS IN THE STATE OF COLIMA, MEXICO

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ABSTRACT

The main purpose of this paper is to analyze and compare different qualitative methodologies to assess the seismic vulnerability of historical constructions in the State of Colima, which is located in a seismic area. The vulnerability is assessed by empirical methods, including the vulnerability class method (VCM) and the vulnerability index method (VIM), in order to perform preliminary indicators of expected damage levels that allow the local authorities to take measures oriented to disaster prevention. Results from the assessment using both methodologies of fifteen historical masonry buildings, most of them from XIX century, are compared with a real vulnerability index of every building from observed damage after the 2003 M7.6 Colima earthquake, according to the classification of damage in masonry buildings (EMS-98).

1. INTRODUCTION

Most of the historical constructions located in the State of Colima, in Mexico, are churches, mainly built (or re-built) in the XIX century. They have the same colonial typology (see figure 1), with variations in size and architectural sophistication. The local society has interest into preserve this cultural patrimony with its original characteristics, due to the architectural and historical importance that these buildings deserve.

In the seismological context, Colima distinguishes by its important exposure, being considered one of the Mexican states under most significant seismic hazard. The historical constructions belong to the groups more vulnerable to earthquakes, as demonstrated by the great damage suffered by this kind of constructions during the earthquake occurred at January 21, 2003 (magnitude 7.6). The Government of Mexico had to invest in expensive works of restoration and rebuilding, generating a restitution of the buildings' structural capacity and, in some cases, increasing their strength. Nevertheless, the safety level of each historical building, repaired or not after the 2003 earthquake, and the possible damage scenario at the occurrence of a larger magnitude earthquake, is completely unknown. Due to these circumstances, it is necessary to execute studies in order to know the seismic risk at the Colima State, and to assess the seismic vulnerability of the historical buildings. The final objective of these studies is to obtain indicators of expected damage levels that allow the local authorities to take measures oriented to disaster prevention.

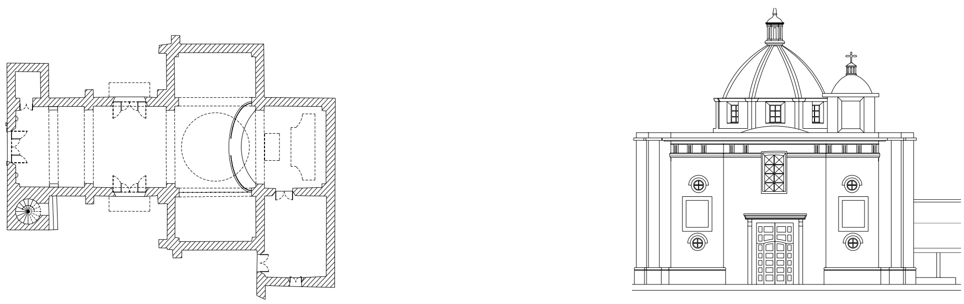

Figure 1. Plan view and façade of a typical Colima church

2. METHODOLOGY

As a first approach, the seismic vulnerability of fifteen historical masonry buildings was assessed in a qualitative way by empirical methods. The basis of these methods is the past experience about seismic behavior of different building typologies, and the characterization of potential structural deficiencies. The empirical methods include the vulnerability class method *(VCM)* and the vulnerability index method *(VIM)*.

The classification of structures used in the *VCM*, was the European Macroseismic Scale (EMS-98) [5]. Table 1.1 presents an EMS-98 summary, considering unreinforced masonry only. This proposal assigns a vulnerability class to every type of structure and is considered as an efficient technique to assess the seismic vulnerability in a quick and satisfactory way. The EMS-98, classifies the different structural typologies in six vulnerability classes going from A to F, in function of the constructive materials used in the building (masonry, concrete, steel, or wood) and the level of seismic design. The first three classes A, B and C, represents the most vulnerable structural typologies (poor performance under earthquakes), such as walls made of adobe, rammed soil, mud and horizontal elements of wood, unconfined masonry walls made of natural stone, brick of fired clay, and frames made of reinforced concrete that were built without a seismic design. The D and E classes represent the medium vulnerable structural typologies (structures with seismic design), into this classification are well-built wooden structures, steel and reinforced concrete, confined and reinforced masonry. Finally the F class represents the less vulnerable structural typologies (structures with a high seismic design).

Table 1.1. Classification of structural typologies according to their vulnerability class. (Summary of the EMS-98, considering only unconfined masonry).

Material	Structural resistant system	Subtypes	Vulnerability Class					
			A	B	C	D	E	F
Masonry	Walls made of natural stone	Walls made of rubble and mortar of mud	X					
		Walls made of stone and mortar of cement or hydrated lime			X			
	Walls made of soil (mud, adobe, and rammed soil)	Walls made of mud	X					
		Walls made of mud with horizontal elements of wood		X				
		Walls made of adobe	X					
		Walls made of rammed soil	X					
	Walls made of brick of fired clay	Unconfined walls and mortar of mud, cement or hydrated lime		X				
		Unconfined walls and mortar of mud, cement or hydrated lime (vertical and horizontal elements of wood)		X				
		Unconfined walls and mortar of mud, cement or hydrated lime (with slabs of concrete)			X			

The *VIM* used in this work is based on GNDT [4], for unreinforced masonry buildings. This method allows the user to identify and characterize the potential structural deficiencies of a building, attributing numerical values (points) to each significant component, and determining, finally, a seismic vulnerability index. The GNDT method has been widely used in Italy during the last years, and it has been upgraded as a result of the continuous experimentation, resulting in an extensive database of damage and vulnerability. The parameters showed in Table

1.2, were compiled in a questionnaire to be applied during the field research. Based in past experiences, the questionnaire have suffered modifications, an example of this is the questionnaire developed by Aguiar et. Al. [1].

In this participation, a base questionnaire developed by Aguiar et. Al [1] was used. However, further modifications were proposed in order to assess buildings under particular conditions. Those modifications consisted particularly in:

Parameter 3: corresponding to conventional resistance, the proposal by Astroza et. Al [2] was adopted.

Parameter 4: soil types were adjusted to Mexican typical soil types (I, II, and III).

Parameter 7: Configuration of elevation. The ratio between total high (T) and bell tower high (H) was used to assign a vulnerability index:

A) $T/H > 0.5$ B)$0.3<T/H<0.5$ C)$0.2<T/H<0.3$ D)$T/H<0.2$

Parameter 9: Typology of the roof. The possibility of Vaults was included

Table 1.2. Vulnerability index numerical scale (I_v) for unreinforced masonry buildings, [3].

i	**Parameter**	***Ki A***	***Ki B***	***Ki C***	***Ki D***	***Wi***
1	Organization of the resistant system	0	5	20	45	1.0
2	Quality of the resistant system	0	5	25	45	0.25
3	Conventional resistance	0	5	25	45	1.5
4	Position and foundation	0	5	25	45	0.75
5	Horizontal diaphragms	0	5	15	45	1.0
6	Floor configuration	0	5	25	45	0.5
7	Configuration of elevation	0	5	25	45	1.0
8	Maximum separation between walls	0	5	25	45	0.25
9	Typology of the roof	0	15	25	45	1.0
10	Non structural elements	0	0	25	45	0.25
11	Conservation level of the building	0	5	25	45	1.0

The use of Table 1.2 is simple, during the research in field is easy to choose one of the four classes A, B, C, or D, (A: Optimal, D: Awful). To every class corresponds a numerical value *Ki* varying between 0 and 45. Also, every parameter is affected for a coefficient of importance *Wi* varying between 0.25 and 1.5. This coefficient reflects the importance of each parameter inside the resistant system of the building, according to the opinion of experts. Next, the seismic vulnerability index (I_v), can be assessed with equation (1).

$$I_v = \sum_{i=1}^{11} K_i \cdot W_i \tag{1}$$

Table 1.3. Ranks to assign the vulnerability class.

Rank	Vulnerability
$Iv < 15\ \%$	Low
$15\ \% \leq Iv < 35\ \%$	Medium
$Iv \geq 35\ \%$	High

Analyzing equation (1), it can be concluded that the vulnerability index defines a scale of values from 0 to the maximum value 382.5. It is divided by 3.825 to obtain a normalized value of vulnerability index, being the rank $0 < I_v < 100$. For a better interpretation of the results, the following ranks are represented as shown in Table 1.3, to assign a vulnerability class to each building.

3. RESULTS

Seismic vulnerability assessment by empirical methods was carried out in fifteen historical buildings in the State of Colima using two different approaches, the *VCM* and *VIM*.

3.1. Seismic vulnerability assessment of the buildings using the *VCM*

The application of the *VCM* consisted on a detailed surveying on everyone of the fifteen historical buildings, in order to obtain the vulnerability class related with the structural typology according to Table 1.1. This assessment was developed on the basis of the building's resistant system such as constructive materials, structural resistant system, structural subtypes; assigning to every building one of the vulnerability classes A, B, C, D, E, or F, being A the highest vulnerability class and F the lowest. It is very important to mention that the assessment was carried out taking in account additional information such as plans, constructive materials characteristics, historical analysis, structural description, previous intervention data and building's conservation level.

The vulnerability class results obtained for all of the fifteen historical buildings are illustrated in Table 1.4. Considering that classes A and B belongs to high vulnerability, C and D to medium vulnerability, E and F to low vulnerability, the results showed that thirteen buildings belong to the medium vulnerability interval and the remaining two were in the high vulnerability interval.

Table 1.4. Vulnerability class for every building.

Name of the building	Vulnerability Class
Convent Ruins of San Francisco de Almoloyan	C (Medium)
Chapel of Nuestra Señora de la Asuncion	B (High)
Museum of Regional History of Colima	C (Medium)
Church of San Felipe de Jesus	C (Medium)
Church of Nuestra Señora de la Merced	C (Medium)
Cathedral Basilica Menor de Guadalupe	C (Medium)
Church of *San Pedro Apostol*	C (Medium)
Church of *San Miguel del Espiritu Santo*	C (Medium)
Church of *Sagrado Corazon de Jesus* (Colima City)	C (Medium)
Chapel of *Virgen del Refugio*	C (Medium)
Church of *San Jeronimo de los Santos Angeles*	A (High)
Church of *Sagrado Corazon de Jesus* (Town of Chiapa)	C (Medium)
Church of *San Jose*	D (Medium)
Church of *San Francisco de Asis*	C (Medium)
Church of *Nuestra Señora de la Salud*	D (Medium)

3.2. Seismic vulnerability assessment of the buildings using the *VIM*

Seismic vulnerability assessment by the *VIM* was applied. The procedure consisted on surveying carefully everyone of the fifteen historical buildings, in order to identify and characterize the potential structural deficiencies of the building corresponding to the eleven parameters shown in Table 1.2, assigning to every parameter one of the four classes A, B, C, or D (A: Optimal, D: Awful), attributing numerical values (points) to each significant component to determine with equation (1), the seismic vulnerability index (I_v), and finally, using Table 1.3 to assign a vulnerability class (high, medium or low) according to the ranks.

Four of the eleven parameters can't be evaluated during the surveying in field, these parameters are the conventional resistance, floor configuration, configuration of elevation and maximum separation between walls, to assess them, it is necessary to use computational tools to simplify the work as AutoCad to obtain dimensions, elevations of the building, areas of the floors and vertical structural elements located in the *X* and *Y* direction, separation between walls, etc.

As in the *VCM* assessment, it was necessary to take into account additional information of every building. The vulnerability index (I_v) results obtained for all of the fifteen historical buildings are illustrated in Table 1.5.

Table 1.5. Vulnerability index for every building.

Name of the building	Iv	Iv %	Vulnerability
Convent Ruins of *San Francisco de Almoloyan*	182.50	47.71	High
Chapel of *Nuestra Señora de la Asuncion*	138.75	36.27	High
Museum of Regional History of Colima	86.25	22.55	Medium
Church of *San Felipe de Jesus*	125.00	32.68	Medium
Church of *Nuestra Señora de la Merced*	155.00	40.52	High
Cathedral *Basilica Menor de Guadalupe*	166.25	43.46	High
Church of *San Pedro Apostol*	172.50	45.10	High
Church of *San Miguel del Espiritu Santo*	183.75	48.04	High
Church of *Sagrado Corazon de Jesus* (Colima City)	126.25	33.01	Medium
Chapel of *Virgen del Refugio*	101.25	26.47	Medium
Church of *San Jeronimo de los Santos Ángeles*	136.25	35.62	High
Church of *Sagrado Corazon de Jesus* (Town of Chiapa)	148.75	38.89	High
Church of *San Jose*	162.50	42.48	High
Church of *San Francisco de Asis*	130.00	33.99	Medium
Church of *Nuestra Señora de la Salud*	125.00	32.68	Medium

The results illustrated in Table 1.5. were different than those of the first approach, nine buildings obtained a high vulnerability index, while the remaining six obtained a medium vulnerability index.

Table 1.6. Classification of observed damage according to the (EMS-98), [5].

Name of the building	Damage (EMS-98)	Vulnerability
Convent Ruins of San Francisco de Almoloyan	Grade 3	(High)
Chapel of Nuestra Señora de la Asuncion	Grade 3	(High)
Museum of Regional History of Colima	Grade 2	(Medium)
Church of San Felipe de Jesus	Grade 2	(Medium)
Church of Nuestra Señora de la Merced	Grade 2	(Medium)
Cathedral Basilica Menor de Guadalupe	Grade 3	(High)
Church of San Pedro Apostol	Grade 4	(High)
Church of San Miguel del Espiritu Santo	Grade 3	(High)
Church of Sagrado Corazon de Jesus (Colima City)	Grade 4	(High)
Chapel of Virgen del Refugio	Grade 2	(Medium)
Church of San Jeronimo de los Santos Angeles	Grade 3	(High)
Church of Sagrado Corazon de Jesus (Town of Chiapa)	Grade 3	(High)
Church of San Jose	Grade 3	(High)
Church of San Francisco de Asis	Grade 3	(High)
Church of Nuestra Señora de la Salud	Grade 3	(High)

Grade 1: Negligible to slight damage (No structural damage, slight non-structural damage), Grade 2: Moderate damage (Slight structural damage, moderate non-structural damage), Grade 3: Substantial to heavy damage (Moderate structural damage, heavy non-structural damage), Grade 4: Very heavy damage (Heavy structural damage, very heavy non-structural damage), Grade 5: Destruction (Very heavy structural damage).

For a better interpretation and comparison of the results, the classification of damage according to the (EMS-98) for every building (grade 1, 2, 3, 4 and 5), was classified in three groups, in order to assign a vulnerability class as shown in Table 1.7.

Table 1.7. Ranks to assign the vulnerability class.

Damage (EMS-98)	Vulnerability
Grade 1	Low
Grade 2	Medium
Grade 3, 4 and 5	High

4 CONCLUSION

Seismic vulnerability of fifteen different buildings was evaluated under two different approaches: Vulnerability class method *(VCM)* and the Vulnerability index method *(VIM)*. It is important to mention that a real vulnerability index of every building was also available from the observed damage after the 2003 M7.6 Colima earthquake, according to the classification of damage in masonry buildings (EMS-98), as shown in Table 1.6. The results obtained by both assessment methods were analyzed and it was concluded that *VIM* provided more accurate results. The nine high vulnerability buildings identified by *VIM* are the buildings that suffered stronger damage during the 2003 earthquake. The *VCM* prediction was poor and the authors consider it not trustable. However, the *VIM* allows the user to perform a preliminary vulnerability assessment in a satisfactory and quick way. In a next stage of this research, the buildings that obtained a higher vulnerability index will be assessed again using quantitative methodologies such as a combination of analytical (theoretical) and experimental methods, in order to obtain and compare a different series of methods.

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GEOMETRICAL AND MECHANICAL CHARACTERISTICS OF A SAMPLE OF MASONRY CHURCH BUILDINGS

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ABSTRACT

In this paper some simple parameters, which describe both the whole building and the single macro-elements geometry, are defined and used for a preliminary assessment of the structural behavior of the church and for an orientative prediction of the in plane collapse mechanism of the single macro-elements.

1. INTRODUCTION

Seismic damage surveys carried out in the aftermath of earthquakes have shown that churches are among the most vulnerable building types and that their response can be studied trough a subdivision in structural macro-elements.

In Mele et al. (1999) [2] a two-steps procedure has been proposed to analyse the basilica churches under seismic loads, consisting in a first phase of 3D analysis of the whole structure (in order to study global behaviour, dynamic characteristics, stresses and the demand of elastic resistance for each macro-elements) and in a second phase of non linear analysis of each macro-element by means of FEM procedures and approximated limit analysis.

In this paper some simple parameters, which describe both the whole building and the single macro-elements geometry, are defined and used for a preliminary assessment of the structural behavior of the church and for an orientative prediction of the in plane collapse mechanism of the single macro-elements.

2. CASES OF STUDY

In this paper ten basilica churches are studied: S. Giovanni Maggiore (SGM), S. Giovanni a Mare (SGMR), S. Paolo Maggiore (SPM), S. Ippolisto Martire (SI), S. Maria Vertecoeli (SMV), Sant'Agostino alla Zecca (SAZ), S. Bernardo e S. Margherita a Fonseca (SBM), S. Gennaro all'Olmo e S. Biagio Maggiore (SGO), S. Maria in Donnaromita (SMD) e S. Maria in Monteverginella (SMM). All

buildings are made of tuff masonry and located in Southern Italy. In Figure 1 the architectural plans of the ten basilica churches are shown. For all case studies, a geometrical-dimensional analysis has been carried out both on the whole building and on each macro-element. From the global point of view, the main dimensions (maximum height (H), length (L) and width (B) in plan) are evaluated as shown in Table 1.
For each church it the weight of roofs (W_r) and of walls (W_w) has been also calculated, and the whole weight (W_t) has been obtained by summation of the previous two contributions as shown in Figure 2.

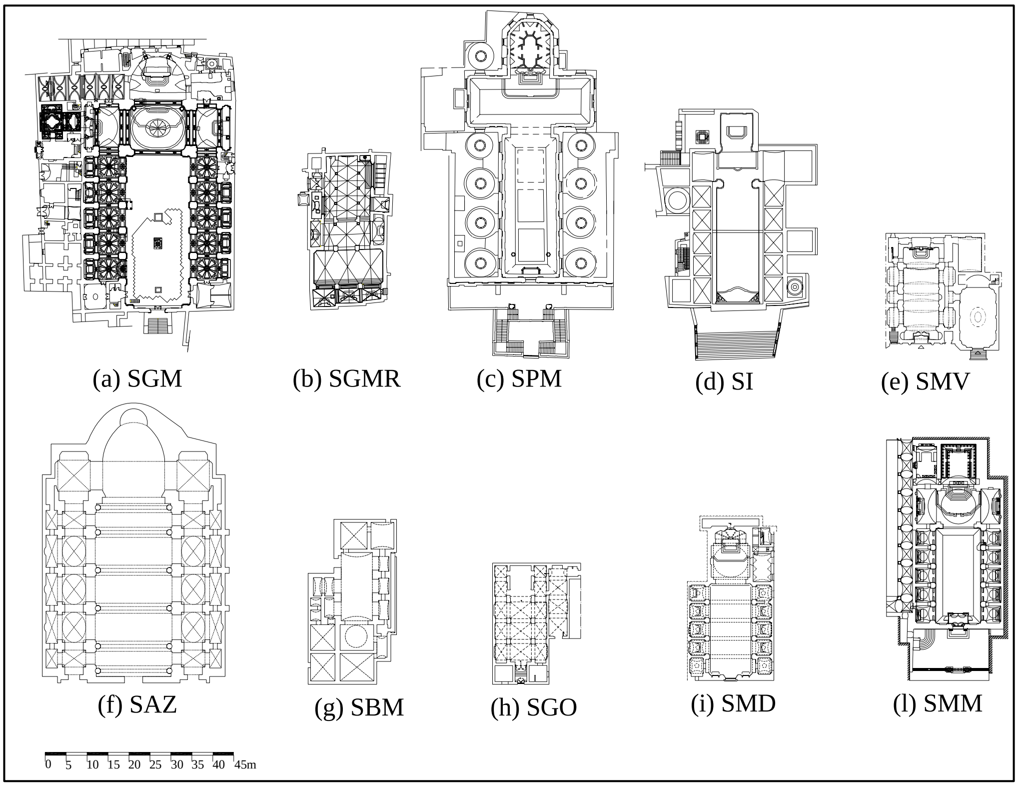

Figure 1: Architectural plan of churches

This first dimensional analysis allows to classify the ten churches into three classes:

1. *Churches of small sizes* (SGMR, SMV, SBM, SMD and SGO), characterized by height around 15m, length 30m and width 20m. Their whole weight is not larger than 50000KN.
2. *Churches of intermediate sizes* (SI and SMM), characterized by height around 20m, length 45m and width 25m. Their whole weight is not larger than 100000KN.
3. *Churches of large sizes* (SGM, SPM and SAZ), characterized by height around 25m, length 65m and width 40m. Their whole weight is larger than 150000KN.

A “linearization” process has been carried out on the architectural plan of the churches, in order to simplify the geometrical description and the subsequent modelling, without significantly altering the structural characteristics (columns and walls sections). From these “linearized” plans the church macro-elements are extracted and labeled with L (for longitudinal walls) and T (for transversal walls), as shown in Figure 3. This approach remarks that from a structural point of view the three-dimensional building is constituted by a wise aggregation of two-dimensional elements.

Table 1:Global Dimensions

	B	L	H
SGM	38	66	26
SGMR	19,5	38	13
SPM	38	66	33
SI	23	47	18
SMV	18	26	15
SAZ	44	66	37
SBM	20	40	14
SGO	13	29	14
SMD	20	37	21
SMM	23	46	21

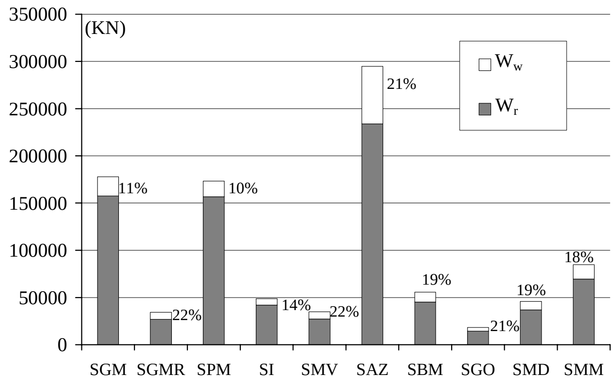

Figure 2: W_t, with W_r and W_w contributions

Eight classes of macro-elements can be defined, on the basis of both the location in the plan and the geometrical characteristics and the architectural functions: (1) *Apse;* (2) *The first triumphal arch; (3) The second triumphal arch; (4) Nave section; (5) Façade; (6)Arcade / Clerestory; (7) Arcade / Clerestory*
The eight classes of macro-elements, for each church, are shown in Figure 4.

3. GLOBAL PARAMETERS

In the following, some geometrical parameters are defined starting from the global dimensions of the churches; such parameters roughly account for some structural properties, and therefore can provide a first simplified assessment of the structural behavior under vertical and horizontal loads. In particular, the ratios B/L, H/B and H/L, respectively accounting for the plan compactness and the maximum and minimum slenderness are computed and given in figure 5, while

the ratios of the wall sections in plan to the global plan area are given in Figure 6. The ratio B/L is approximately equal to 0.5 for all cases, with the exception of the SMV and SAZ churches, where it is equal to 0.7. The maximum slenderness (H/B) is in the range of 0.7 (SGM and SGMR) and 1.05 (SMD), while the ratio H/L is comprised between 0.4 (SGMR and SBM) and 0.6 (SMV and SMD).
The ratio between the area of masonry walls in plan (A_{wtot}) and the whole plan area (A_{tot}) is an index of the building bearing capacity; the Figure 6 shows that this ratio is in the range of 15%÷20%, which corresponds to values of 1/5÷1/6 typical for masonry structures as reported by Rondelet (Rondelet 1802 [5]).

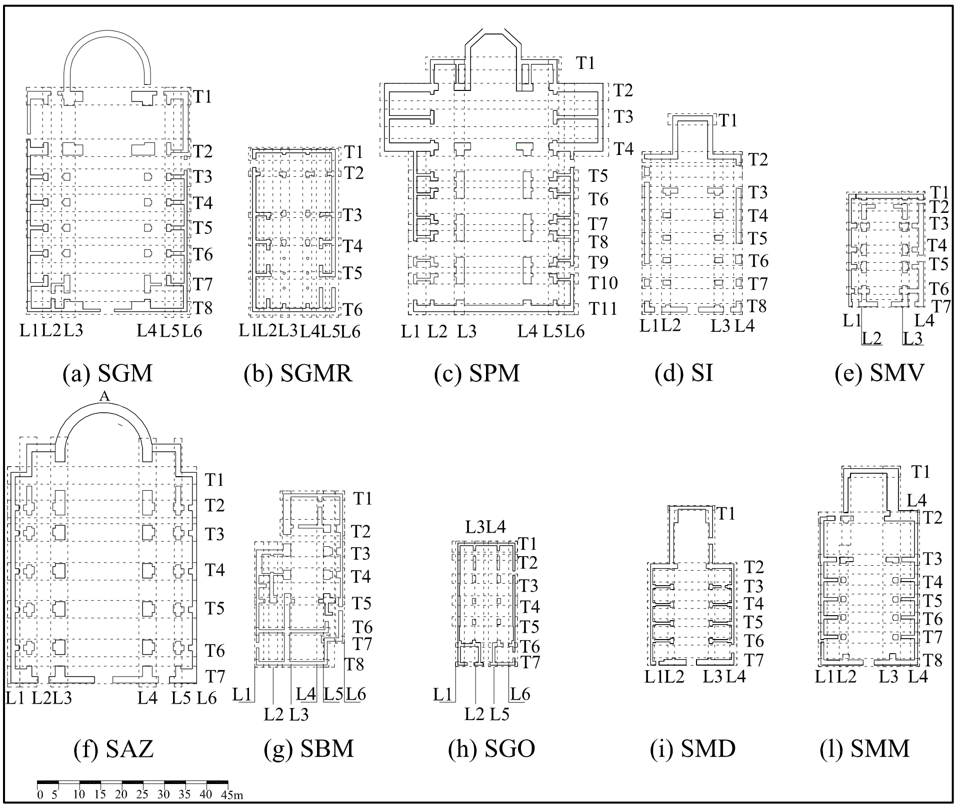

Figure 3: Linearized plans

The exceptions are the churches of SMV and SBM, with slightly larger values (27÷26%), due to the relatively large thickness of the walls.

This ratio is an index of the behavior of the building under gravity loads, since it can be directly related to the average compressive stresses at the base of the building. The average value of the compressive stress at the base of the buildings, σ_{mean}, is computed as the ratio W_t/A_{tot} and given in Figure 7, together with the maximum and minimum values of base compressive stress. This figure confirms the relation between the simplified parameters A_{wtot}/A_{tot} and the stress value: in fact the church of SMV and SBM, which presents the highest values of $A_{wtot/}$Atot are characterized by the lowest values of the average stress at the base. The values of average stress are quite uniform for all churches, and falls in the range 0.3÷0.5MPa, while the minimum stress is comprised between 0.1÷0.3MPa. On the contrary, the maximum stress shows a larger variation, with values comprised between 1.3MPa, for the church SGMR, and 0.3MPa, for the church SMV. An additional geometrical parameter, i.e.: the ratio between the area of masonry walls, in each principal direction of the plan, and the whole plan area ($A_{w,transv}/A_{tot}$ and $A_{w,long}/A_{tot}$) is computed for the ten churches and given in figure 8.This ratio is related to the structural behavior under lateral loads and provides a measure of the relatively shear strength and stiffness of the building in the two principal directions. In fact the seismic codes suggest minimum values for such parameters, as a function of the number of stories and of the seismic zone; such limitation is equal to 2÷5% in the EC8 [4], while according to the Italian seismic code OPCM 3431([3]), it is 3.5÷7%; more stringent limitations, equal to 10%, are suggested by Lourenço et al. (2005) [1] for historic building in high-seismicity zone. Therefore, by adopting the latter limitation suggested by Lorenço, it can be observed that only the SPM, SMV, SAZ, SBM and SMM churches satisfy the above criterion in both directions. Moreover it can be noticed that in the cases of SGMR, SMV, SAZ, SBM, and SGO there is a significant difference between the percentages in the two directions, while this difference is less evident for SGM,SPM,SI,SMD, and SMM churches. It is important to observe that the above geometrical parameters can give a significant measure of the resistant capacity of the churches only in the case that the structural elements are characterized by a shear failure mechanism, which is typical of stocky-solid walls; on the contrary, the macro-elements of the churches are frequently characterized by columns, large openings and arcades, which cause a frame-type collapse mechanism. For this reason it seems very important to examine the geometry of the single macro-elements of the churches by defining the openings percentage ($(A_o/A_t)_{macro}$) and the ratio between the height and the width of the wall (h/b). The first parameter suggests if the wall is characterized by a frame mechanism, while the second one suggests if the wall collapse is due to shear or overturning failure.

4. GEOMETRICAL PARAMETERS OF MACRO-ELEMENTS

The observations of seismic damage on masonry structures suggest that the masonry walls can be characterized by collapse mechanism either in-plane or out-of-plane. It has been defined also the geometrical parameters which can be used to predict the collapse mechanism typology in the plane of the wall. In particular, the openings percentage $(A_o/A_t)_{macro}$) can be considered as an index which can suggests if the macro-element is characterized by a frame mechanism, (large values of the opening percentage) or by solid wall mechanisms, (low values of $(A_o/A_t)_{macro}$). In the last case the ratio h/b (height / width of the wall) supplies useful indications on the possible mechanism: in particular for h/b›1.5, an overturning failure can be expected; for h/b‹1 a shear-sliding failure

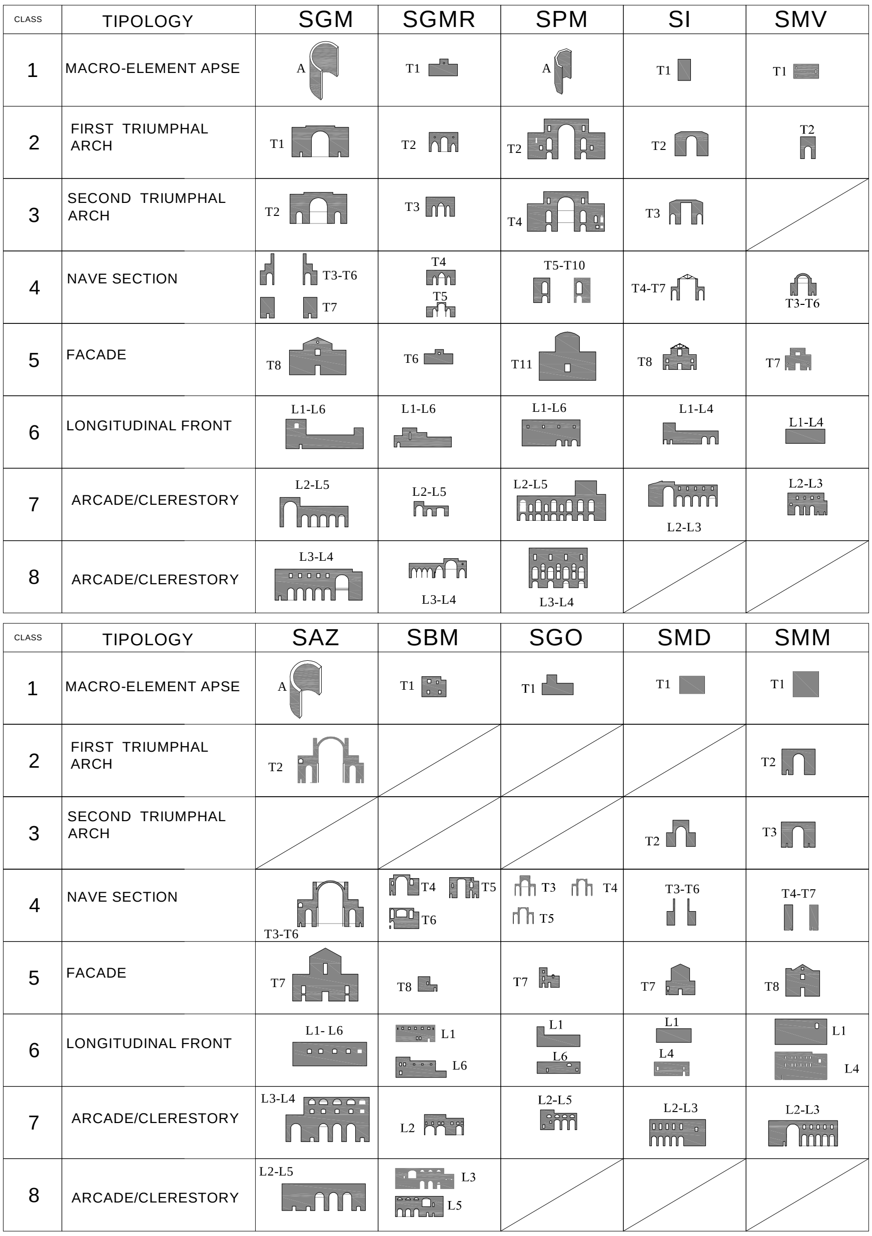

Figure 4: Classes of Macro-element

mechanism is likely to occur; for h/b between 1÷1.5 the mechanism can vary as a function of the gravity load and of the mechanical properties of masonry.
The Figure 9 shows the values of $(A_o/A_t)_{macro}$ which have been calculated for the most significant classes of macro-elements in the ten churches, i.e. the classes 2,4 and 5 for the transversal direction , and classes 6 and 7 for the longitudinal direction.

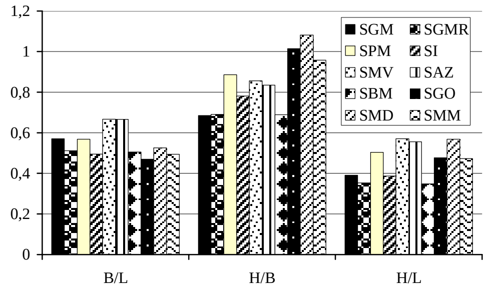

Figure 5: Ratio of global dimension

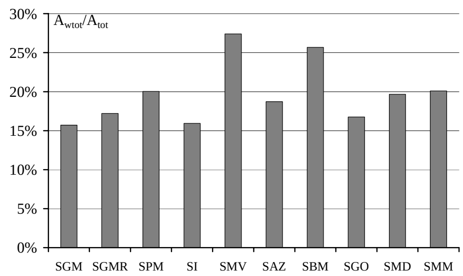

Figure 6: Ratio of A_{wtot}/A_{tot}

It is possible to observe that the class 2 (first triumphal arch) are characterized by an opening percentages respectively of 25÷35% (with exception of SPM and SAZ which presents percentages respectively of 58% and 10%); for the macro-elements of class 4 (nave section) the percentage is in the range of 40÷70%. In the case of longitudinal macro-elements it can be observed that for the classes 5 (façade) and 6(longitudinal front) the value of the opening percentage is quite uniform, while class 7 (arcade/clerestory) shows larger values and significant variation (20÷40%), with a very high value (55%) for the SGM church.

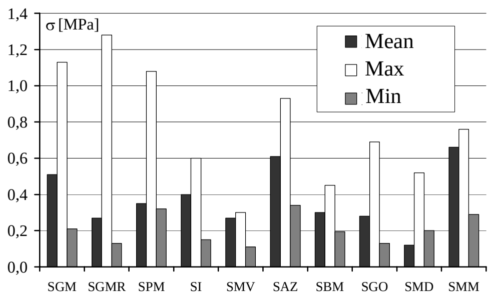

Figure7: Stresses at the base of walls

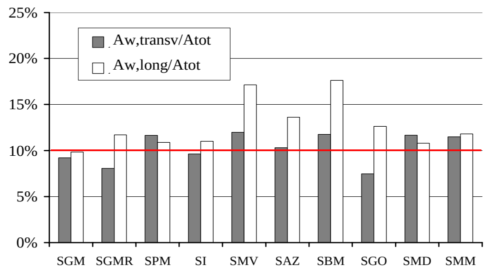

Figure 8: $A_{w,\,transv}/A_{tot}$ and $A_{w,\,long}/A_{tot}$

On the basis of these results we can state that for the macro-elements of the classes 5 and 6, characterized by low values of opening percentage, the in plane collapse under lateral loads is due to a shear or overturning mechanism, while for the other macro-elements classes, with larger value of openings percentage, frame mechanisms are expected. The above indications are confirmed by the results of non linear analyses, not reported here due to page limitations.

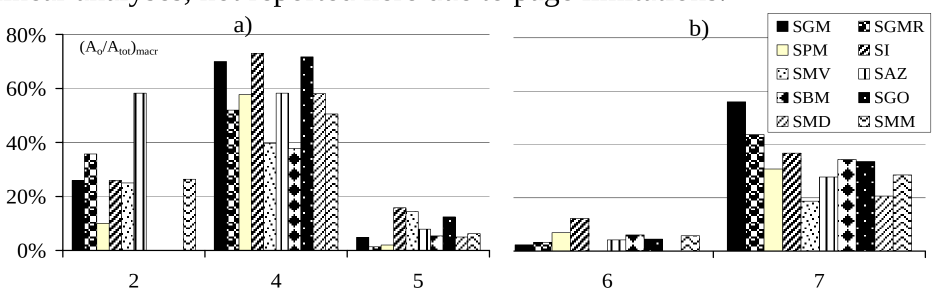

Figure 9: Opening ratio for classes of macro-elements

5. CONCLUSIONS

In this paper some results of a geometrical-dimensional analysis carried out on a sample of masonry church buildings have been presented. Global dimensions have been examined, the weight of the structural elements and of the whole buildings have been calculated. The macro-elements of each church have been extracted from the plan and have been grouped in eight typological classes.

Simplified geometrical parameters are defined to predict the structural behavior under vertical load ($A_{w,tot}/A_{tot}$ related to the basis stresses) and horizontal load ($A_{w,i}/A_{tot}$ related to the seismic vulnerability).

Finally the geometry of each macro-element has been studied defining two indexes ($(A_o/A_{tot})_{macro}$ and h/b) which can be used to predict the collapse mechanism of the macro-element.

ACKNOWLEDGEMENT

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FOUNDATION ANALYSIS OF THE RIALTO BRIDGE IN VENICE

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ABSTRACT

In restoration and consolidation design, knowledge of the interactions between soil and structure plays a major role in structural behaviour over time. This aspect is particularly important when dealing with the city of Venice, where the buildings are generally characterised by significant total and differential settlements, as a result of stratigraphic variability and the pressure induced by varying types of foundation.

The interactions between the construction and foundation as they develop over time must take the building’s history into consideration, making full use of archive sources and on-site inspections.

This paper presents a recent historical reconstruction of the behaviour of the Rialto Bridge foundations, as an example of soil-structure interactions within the framework of critical-state soil mechanics.

1. INTRODUCTION

Venice represents an example of considerable architectural complexity, due to its physical environment, construction materials, and strong connections between stone and water. One of the basic considerations in analysing structural behaviour must begin with foundation type as well as dimensions, materials, and state of decay.

A study of the existing foundations presented serious difficulties as a result of limited information concerning their construction and modifications undergone over time. To carry out this type of analysis, a multidisciplinary approach had to be adopted, to include geotechnical and structural aspects as well as an examination of the building’s history.

The fourth bridge over the Grand Canal in Venice (Santiago Calatrava Valls project) is nearing completion. The project involves deep foundations, composed of diaphragm walls especially designed to absorb horizontal thrusts and reduce lateral movements.

About 400 years ago, during the reconstruction in stone of the Rialto Bridge (1588-1592), the architect Antonio Da Ponte conceived its foundation structures as devices which could reduce horizontal displacements which, compatibly with the building techniques of the times, involved mechanisms similar to those of modern foundations.

This study, following the historical itinerary which led to the building of the Rialto Bridge, applies numerical methods in order to analyse the behaviour of the soil and the foundations of the bridge. In particular, it shows how the architect viewed the foundation-superstructure complex as an efficient mechanism for absorbing both horizontal thrusts and settlements, despite the poor mechanical properties of the underlying soil and also the adverse opinions of well-known architects who were also working in Venice at that time.

2. HISTORY OF THE RIALTO BRIDGE

The first Rialto bridge was built in 1172, first on a line of boats and then with a fixed wooden structure [4, 6] (figure 1). It was originally called the Ponte della Moneta ("coin bridge") "because of the toll which had to be paid for crossing it".

After many and frequent repairs, the wooden bridge collapsed in 1444, was then rebuilt, again in wood. In 1458, "shops" were housed on it, and its name was changed, from Ponte della Moneta to Ponte di Rialto.

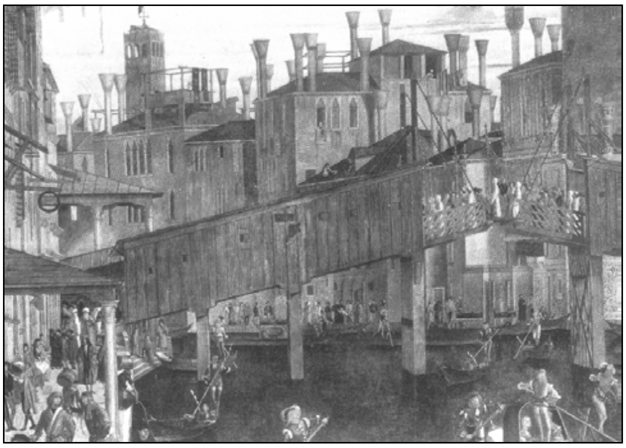

Figure 1 – The wooden Ponte di Rialto. (detail from "*Il miracolo della reliquia della Croce*" by Carpaccio, Gallerie dell'Accademia, Venice, 1494)

In 1524, in view of the precarious condition of the bridge, rebuilding it in stone began to be considered seriously. Many artists were consulted about it, including Andrea Palladio [2] and Jacopo Sansovino, and, during his brief stay in Venice in 1529, Michelangelo Buonarroti also studied a project for it.

In 1587, the decision was made to begin building the bridge in stone [3]. After examining several projects, in January 1588 the Venetian Senate decided to adopt a single-arch solution. Antonio Da Ponte developed the plans (figure 2), a combination of technical and architectural elements from several drawings and plans presented over the course of past years, and was entrusted with the responsibility for the new building. Work began in February 1588.

Particular attention was paid to the foundations, the constructive modalities of which were opposed by influential technicians of the times [5]. Indeed, a special commission of inquiry was established to examine safety aspects, in order to decide whether or not to proceed with the original project.

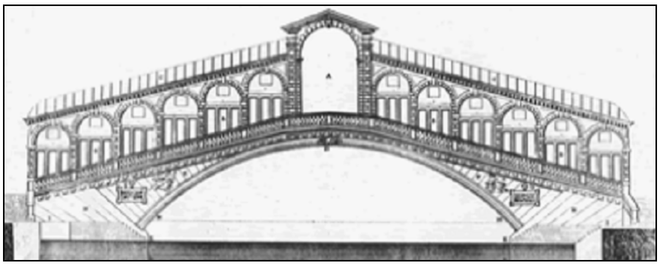

Figure 2 – Plan for the Ponte di Rialto, single-arch solution.
(Antonio Da Ponte, 1588)

Da Ponte showed how the foundations worked by illustrating the presence of buttresses to absorb horizontal thrusts, and the links of the foundations with nearby buildings. The result of the commission's inquiry was favourable to Da Ponte, who finished the new stone bridge over the Grand Canal in January 1592 (figure 3).

Figure 3 - The Rialto Bridge, aerial view.

3. GEOTECHNICAL MODEL AND RIALTO BRIDGE FOUNDATIONS

The stratigraphy in lagoonal environments, and especially in the city of Venice (figure 4), is composed of a top layer of more or less recent man-made material 2-3 metres thick. Underneath are mainly cohesive layers, of soft to medium consistency, sometimes with soft, compressible, organic soil. These layers are followed by alternating compact or medium-consistency clay with sandy silt and fine sand. Overconsolidated silty clay layers (called *caranto*) are sometimes found.

The constitutive models used for finite-element modelling of the mechanical behaviour of these soils are of two main types:

- elasto-plastic model (Mohr-Coulomb failure criteria and non-associated flow law), for the behaviour of granular materials;
- isotropic hardening model (modified Cam-Clay), for the behaviour of cohesive materials.

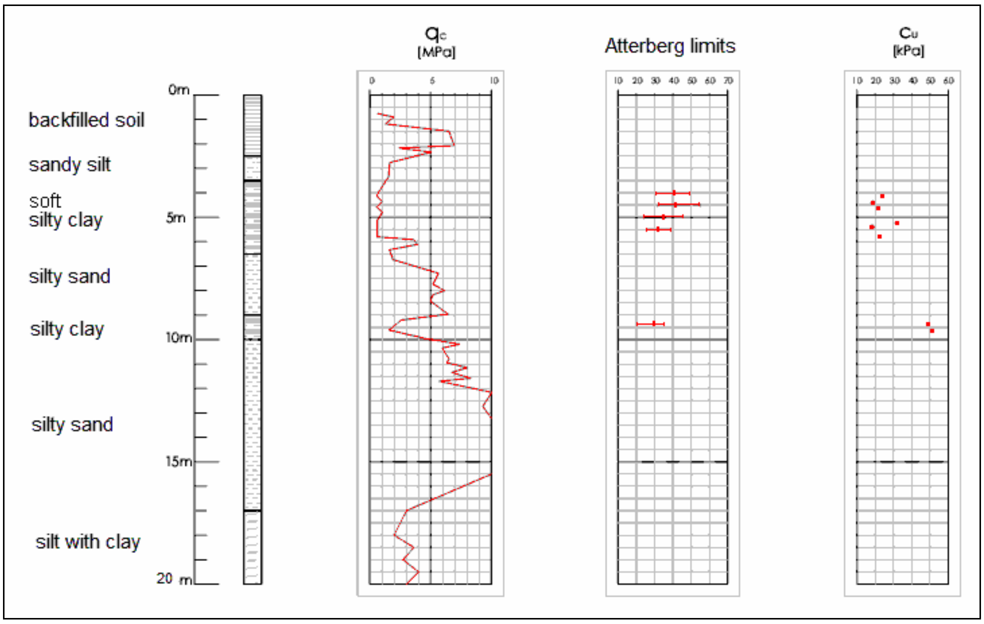

Figure 4 - Soil profile of the Rialto Bridge site.

The minutes of the commission of inquiry of 1588 were analysed, so that a detailed reconstruction of the Ponte di Rialto foundations could be made. In particular, operations to consolidate the soil near the abutments and reinforcements of the foundations of nearby buildings such as the Palazzo dei Camerlenghi and the Drapperia, to contrast the horizontal thrust of the bridge (figures 5, 6, 7), were highlighted. The same constructive criterion was also adopted for the foundations on the San Marco side.

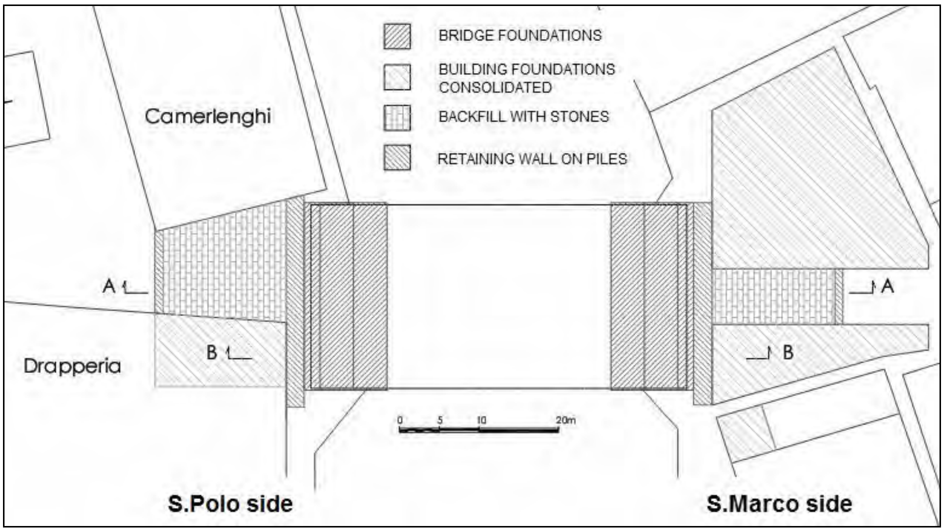

Figure 5 – Plane view of the Rialto Bridge foundations.

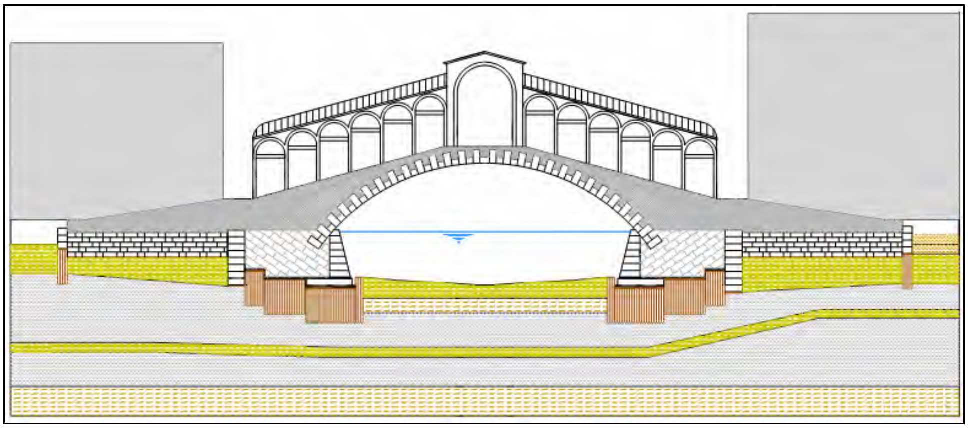

Figure 6 – Section view A-A.

In order to avoid damage to the foundations of the Drapperia and the Palazzo dei Camerlenghi, a retaining wall was also built (figure 7).

Figure 8 shows the reconstruction of the load history on the foundation base.

4. RESULTS OF ANALYSIS

Finite-element simulation of soil-structure interactions was used to analyse the evolution of settlements undergone by the bridge in time, and it was found that deformation were totally absorbed by the foundations, considered as a whole [1].

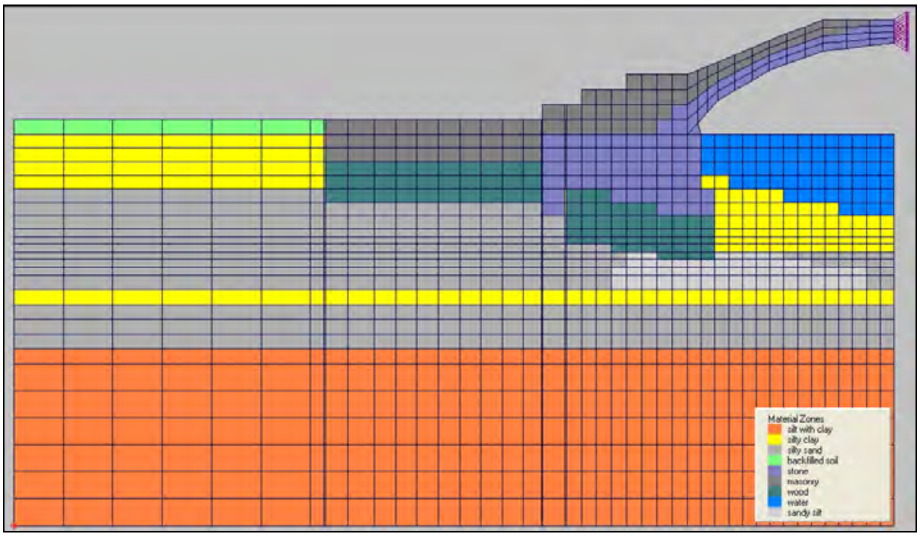

Figure 7 – Mesh detail of sectional view B-B (elements: quadrilateral 8 nodes).

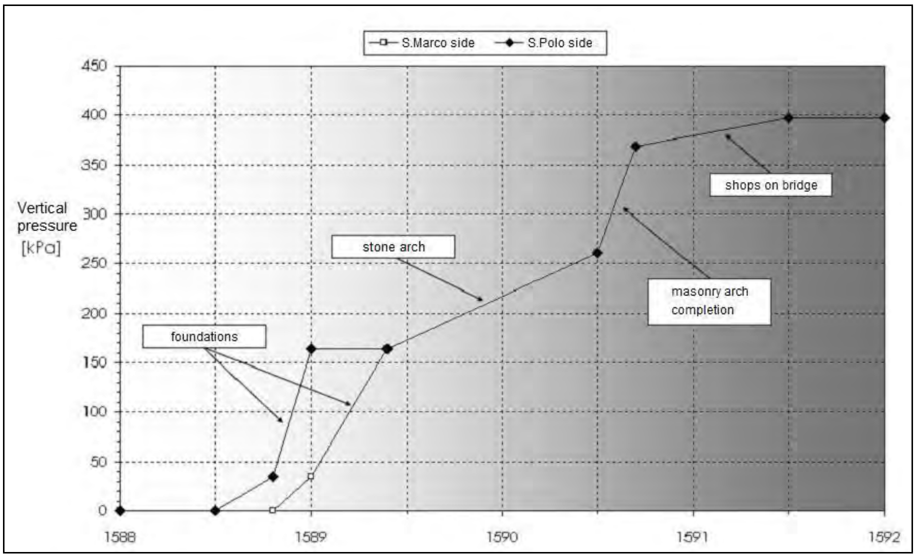

Figure 8 - Reconstruction of load history on the foundation base.

In other words, the bridge structure and the stone arch followed the vertical movements of the foundations, whereas horizontal stresses were not particularly high. In effect, at every point belonging to the vault, the differential of horizontal movements is about 0.1 cm, and horizontal movements of the whole foundation about 1 cm (figure 9).

Results of analysis show how the structure chosen by Da Ponte was kinematically capable of associating the vertical movements of the abutments with slight rotations, thus maintaining the arch under compression and avoiding the dangerous horizontal displacements foreseen by some technicians of the time.

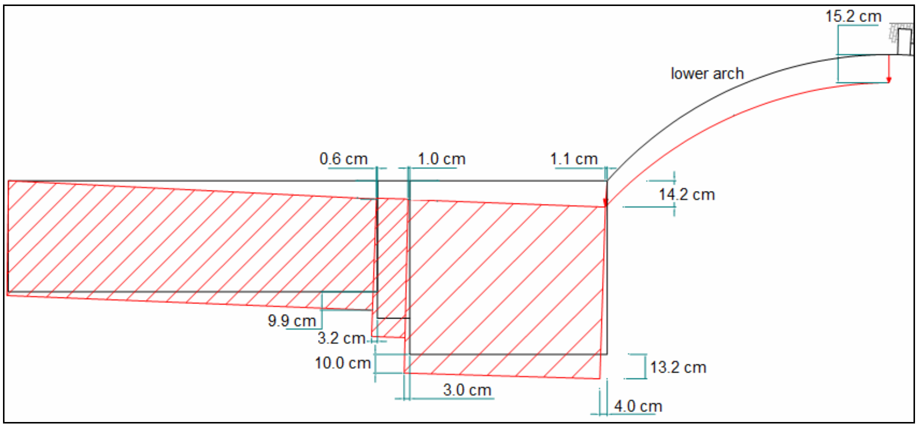

Figure 9 – Settlement values of bridge and nearby buildings.

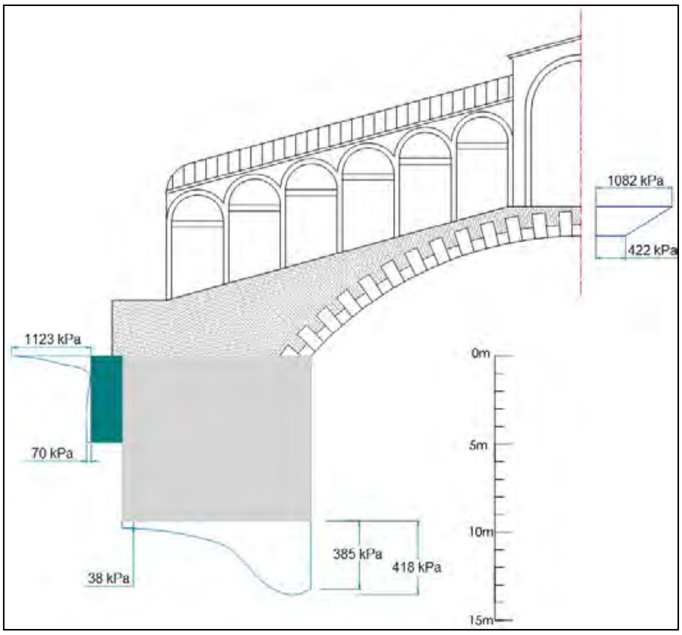

Figure 10 – Stresses on the foundation base, foundation buttress and bridge vault.

Analysis of the stresses (positive = compression) acting on the bridge (figure 10) shows that pressure is greatly concentrated on the abutment, due to the buttress. This is due to the presence of the retaining wall so that the bridge can discharge some of its stresses (about 875 kN/m) on the foundations of the Drapperia. The effect of the buttress may also be noted in the keystone of the vault, where the entire section is compressed (1311 kN/m) but, in its absence, the values are 922 kN/m in compression and 40 kN/m in traction, which may generate vault cracking. Similar results are obtained when considering the reinforcement of the central portion of the bridge abutments with backfilled stones (section A-A).

Consolidation works and buttress have meant that the stone structure has remained intact in the course of centuries, and that the formation of dangerous traction phenomena has been avoided. This has been confirmed by several inspections of the vault structure. It is clear that the old Venetian builders knew very well that movements in stone structures could cause splintering and fracture of the stone material itself.

It may thus be stated that the operations chosen by Da Ponte were essential to protect both the bridge and adjacent buildings.

5. CONCLUSIONS

On the base of a recent analysis of historical documents reporting the reconstruction "in stone" of the Rialto Bridge, with particular reference to the type of foundations, it was possible to follow in detail the various stages of construction and to study the behaviour of the consolidation works which allowed the bridge to function correctly. These aspects have always been neglected in the literature on the subject, which has never demonstrated exactly how Da Ponte resolved the problem of horizontal thrust.

The correct approach for studying foundation works in the city of Venice involves various aspects involving definition of load distributions. In addition, historical investigation, providing details of the construction method and materials used, allows us to understand how the building developed over time.

Such data and detailed geotechnical investigations made study of soil-structure interactions possible, and defined proper preservation and restoration operations.

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COLLAPSE OF MASONRY ARCHES OF DIFFERENT SHAPES UNDER CONSTANT LATERAL ACCELERATION

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ABSTRACT

Masonry arches are typical components of historic buildings throughout the world, and their damage or collapse is very often caused by earthquakes. This paper analyses the stability of masonry arches and portals subjected to constant horizontal acceleration, combined with the vertical acceleration due to gravity. It focuses on pointed and basket-handle arches, in order to obtain information on the relative stability of these shapes for varying geometry parameters. The equivalent static analysis determines the value of the constant lateral acceleration needed to cause collapse of the arch, which coincides with the minimum peak ground acceleration needed to transform the arch into a mechanism.

1. INTRODUCTION

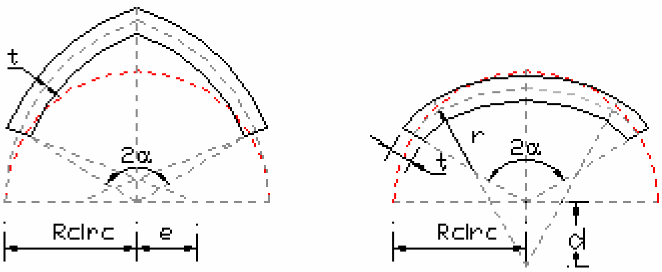
Unreinforced masonry structures, of which arches are typical elements, are a large part of the world architectural heritage. These structures are particularly vulnerable to seismic events, as shown by the recent damage of invaluable monuments worldwide. Most research conducted so far on arches and portals under horizontal accelerations has followed an equivalent static approach [1],[2],[3],[5]. With this approach the arch or portal is considered subjected to a constant horizontal acceleration and a vertical one due to gravity. The analysis computes the minimum work needed for the formation of a sufficient number of non-dissipative hinges transforming the arch into a mechanism. The studies conducted thus far have focused on semicircular arches and portals. Conversely, there is a lack of information about arches and portals of different shape, such as pointed and basket handle arches. Pointed arches, typical of the Gothic architecture, allowed the Gothic cathedrals to reach larger heights than the Romanesque ones, while bearing lower thrusts for given loads and spans and reducing the weight on the lateral walls. The basket handle arch, for a given span, generates a lower opening. Its principal application was in bridges, though there are also examples of basket handle arches in portals and in masonry buildings.

In this paper the equivalent static analysis, based on the geometry and independent of the dimensional scale, is used to determine the minimum constant lateral acceleration for collapse of masonry arches and portals of pointed and basket-handle shape.

2. METHOD OF ANALYSIS

Under the usual assumptions of limit analysis [4], masonry becomes an assemblage of rigid ashlars which are not able to slide or crush, but can only disconnect forming non-dissipative hinges. At incipient collapse, a general shaped arch subjected to an horizontal constant acceleration will be transformed into an asymmetric mechanism, with formation of four hinges.

This paper considers two types of arches, namely pointed and basket-handle arches, illustrated in Figure 1. In order to study the comparative stability of these shapes, their main geometry parameters are here defined in dimensionless form: the t/R_{circ} ratio between the arch thickness t and the centerline radius of the reference circular arch R_{circ}; the angle of embrace 2 ; for pointed arches, the e/R_{circ} ratio between the eccentricity of the arch center e and R_{circ}; for basket-handle arches, the ratios r/R_{circ} and d/R_{circ}, r being the smaller radius and d the distance of the middle center from the horizontal springline of the reference circular arch (Figure 1).

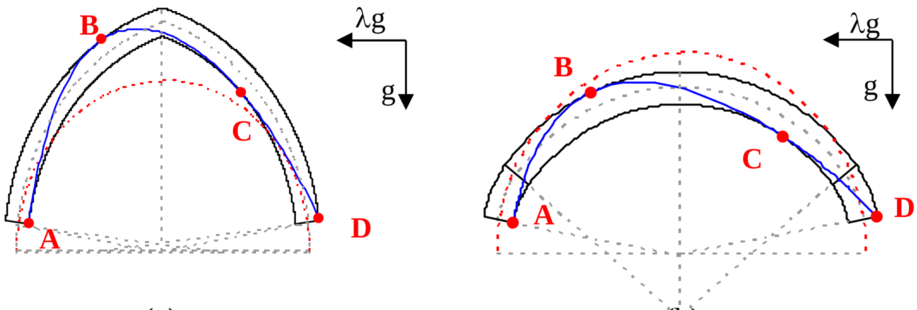


(a) (b)

Figure 1 – Geometry of (a) the pointed and (b) the basket-handle arch

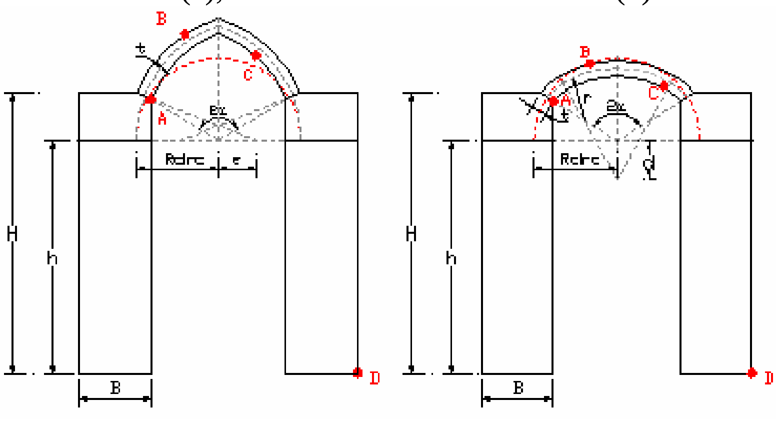
Collapse of the arch requires four non-dissipative hinges to form, two at the extrados (B and D, Figure 2), and two at the intrados (A and C). For all kinematically admissible positions of the hinges, a corresponding value of constant lateral acceleration can be computed applying equilibrium in the form of the principle of virtual work. The real hinge locations correspond to the lowest value of lateral acceleration, here indicated as g (where g is the acceleration of gravity), and this is the constant acceleration leading to collapse of the arch. Hence, and the hinge locations at collapse must be found by iterative calculations. When the mechanism forms, the line of thrust is contained within the thickness of the arch and is tangent to the boundary in the four hinge points.

The same analysis is then extended to the portal (i.e. to the arch on buttresses) considering the additional parameters B/R_{circ} (B = width of the buttress) and h/R_{circ} (h = height of the buttress from the base to the horizontal springline of the reference circular arch). Collapse of a portal also requires four hinges to form, two at the extrados (B and D in Figure 3), and two at the intrados (A and C). For the generic portal, three types of mechanism can be activated: a local mechanism, characterised by the local failure of the arch only, without any hinge occurring in the buttresses; a global mechanism, characterised by the presence of two hinges at the base of the buttresses and two hinges in the arch; and a semi-global mechanism, characterised by the presence of one hinge at the base of one buttress and three hinges in the arch [3]. The cases in Figure 3 refer to a semi-global mechanism of collapse.



(a) (b)

Figure 2 – Example of hinge formation and line of thrust at collapse in the pointed arch (a), and in the basket-handle arch (b)



(a) (b)

Figure 3 – Example of hinge locations at collapse in the portal with pointed arch (a) and basket-handle arch (b) (semi-global mechanism)

3. RESULTS FOR MASONRY ARCHES

The analysis was carried out on arches with an angle of embrace 2 equal to 120°, 150° and 180°, varying the ratios e/R_{circ} and t/R_{circ} for pointed arches, and d/R_{circ} and t/R_{circ} (with r/R_{circ} fixed at 0.5) for basket-handle arches. As follows, the lateral stability of the arches is expressed in terms of $/ _{circ}$, where g is the horizontal acceleration at collapse of the arch under consideration, and $_{circ}$g is that of the reference circular arch.

Pointed arches with 2 = 120° are always less stable than the reference circular ones, and their stability decreases as the e/R_{circ} ratio increases (Figure 4-a). The relative stability $/ _{circ}$ is very weakly influenced by the thickness of the arch. Similar results are found for 2 = 150° (Figure 4-c), but the decrease of stability with e/R_{circ} is less pronounced and more influenced by t/R_{circ}. Thinner arches are more stable (in relative terms to their circular counterparts) than thicker arches. For 2 = 180° (Figure 4-e), thin pointed arches may be more stable than their reference circular arches. In this case, for each given t/R_{circ} ratio, the collapse acceleration reaches the maximum value for a certain e/R_{circ}, which can be defined as the optimal ratio (Figure 4-e). The maximum value of collapse acceleration decreases as the thickness ratio increases. The optimal e/R_{circ} decreases as the t/R_{circ} ratio increases, until it reaches zero (circular arch) for the largest thickness ratio analyzed (0.20).

Basket-handle arches with 2 = 120° are always more stable than the reference circular ones, and their stability increases as the d/R_{circ} ratio increases (Figure 4-b). Thinner arches are less stable (in relative terms to their circular counterparts) than thicker arches. Similar results are found for 2 = 150° (Figure 4-d), but the curves corresponding to the smallest thickness ratios start displaying a maximum point. For 2 = 180° (Figure 4-f), basket-handle arches with the smallest analyzed thickness ratio (0.12) have a large $/ _{circ}$ for moderate d/R_{circ} values, but become less stable than their reference circular arches for large d/R_{circ} values. For each given t/R_{circ} ratio, the collapse acceleration reaches the maximum for a certain d/R_{circ} ratio (Figure 4-f). The maximum value of collapse acceleration decreases as the thickness ratio increases. The optimal d/R_{circ} increases as t/R_{circ} increases.

For given shape parameters, arches with smaller angles of embrace and larger thickness ratios were found to be more resistant to lateral accelerations. For angles of embrace under a critical value, the collapse mechanism is characterized by the formation of two hinges at the supports, and other two hinges within the span. For angles of embrace above this critical value, three hinges form within the span, and only one hinge forms at the support. These results all agree with previous results on the circular arch. No graphs can be provided due to space limitations.

4. RESULTS FOR MASONRY PORTALS

The analyses illustrated above for the stand-alone arches were repeated for portals having arches of pointed and basket-handle shapes, in order to study the influence of the e/R_{circ} and d/R_{circ} ratios on the collapse accelerations. Portals with 2 equal to 120°, 150° and 180°, and with h/R_{circ}=3 and B/R_{circ}=0.8 were analyzed.

Portals with pointed arches having 2 = 120° and 150° are always more stable than their circular counterparts, although $\tilde{\ }_{circ}$ is rather small (Figures 5-a, c). For each t/R_{circ}, the collapse acceleration reaches a maximum value for a certain e/R_{circ} ratio, which can be considered as the optimal ratio in terms of stability. This value increases as the t/R_{circ} ratio increases. In general, thinner arches are less stable (in relative terms to their circular counterparts) than thicker arches. For 2 = 120°, the semi-global mechanism always controls. As 2 increases, the local mechanism controls for a wider range of e/R_{circ} values, and especially so for small thickness ratios (Figures 5-c,e), until it becomes the only mechanism for 2 = 180° and the three smallest thickness ratios analyzed.

When 2 = 180° and failure is by a local mechanism, thinner arches may become more stable (in relative terms to their circular counterparts) than thicker arches, as previously observed for stand-alone arches, although this does not hold for all thickness ratios. The values of $\tilde{\ }_{circ}$ become much larger than one for thickness ratios between 0.12 and 0.14 and “moderate” e/R_{circ} values.

As opposed to stand-alone pointed arches, portals with pointed arches are in most cases more resistant to lateral accelerations than those with circular arches. This holds both when semi-global and local collapse mechanisms control (Figure 5-e) except for some combinations of thickness ratio and e/R_{circ}.

Portals with basket-handle arches having 2 = 120° and 150° are always less stable than their circular counterparts, and the stability decreases as the d/R_{circ} ratio increases (Figures 5-b, d). Thinner arches are more stable (in relative terms to their circular counterparts) than thicker arches. For 2 = 120° and 150°, the semi-global mechanism always controls. For 2 = 180°, the local mechanism controls in some ranges of d/R_{circ} values (Figure 5-f). The values of $\tilde{\ }_{circ}$ become much larger than one for thickness ratios between 0.12 and 0.14 and “moderate” d/R_{circ} values.

When collapse is controlled by a semi-global mechanism, portals with pointed arches show increasing resistance to lateral accelerations as the angle of embrace increases and the thickness ratio decreases. The stability of portals with basket-handle arches versus the angle of embrace displays a minimum, and increases as the thickness ratio decreases. No graphs can be provided due to space limitations. When a local mechanism controls, the collapse acceleration becomes the same of the stand-alone arch and hence displays the trend illustrated in the previous section.

5. CONCLUSIONS

In this paper, the stability of masonry arches and portals of different shapes subjected to a constant horizontal acceleration is investigated. Results of the analysis indicate that the geometry parameters have a pronounced influence on the stability of an arch or a portal. In particular the following main conclusions can be drawn:

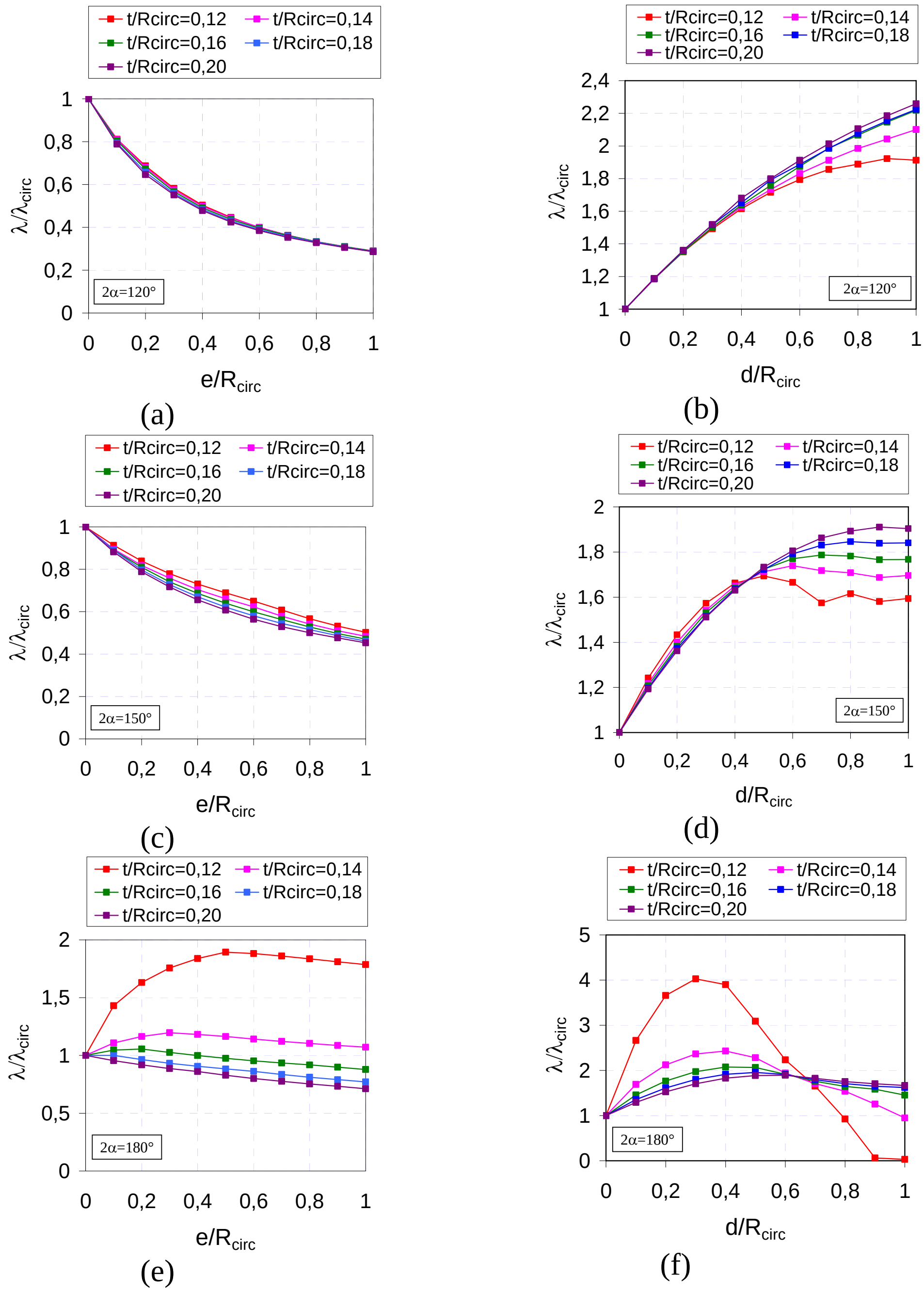


Figure 4 – Lateral stability of pointed (a-c-e) and basket-handle (b-d-f) arches with angles of embrace 2 equal to 120°, 150° and 180°.

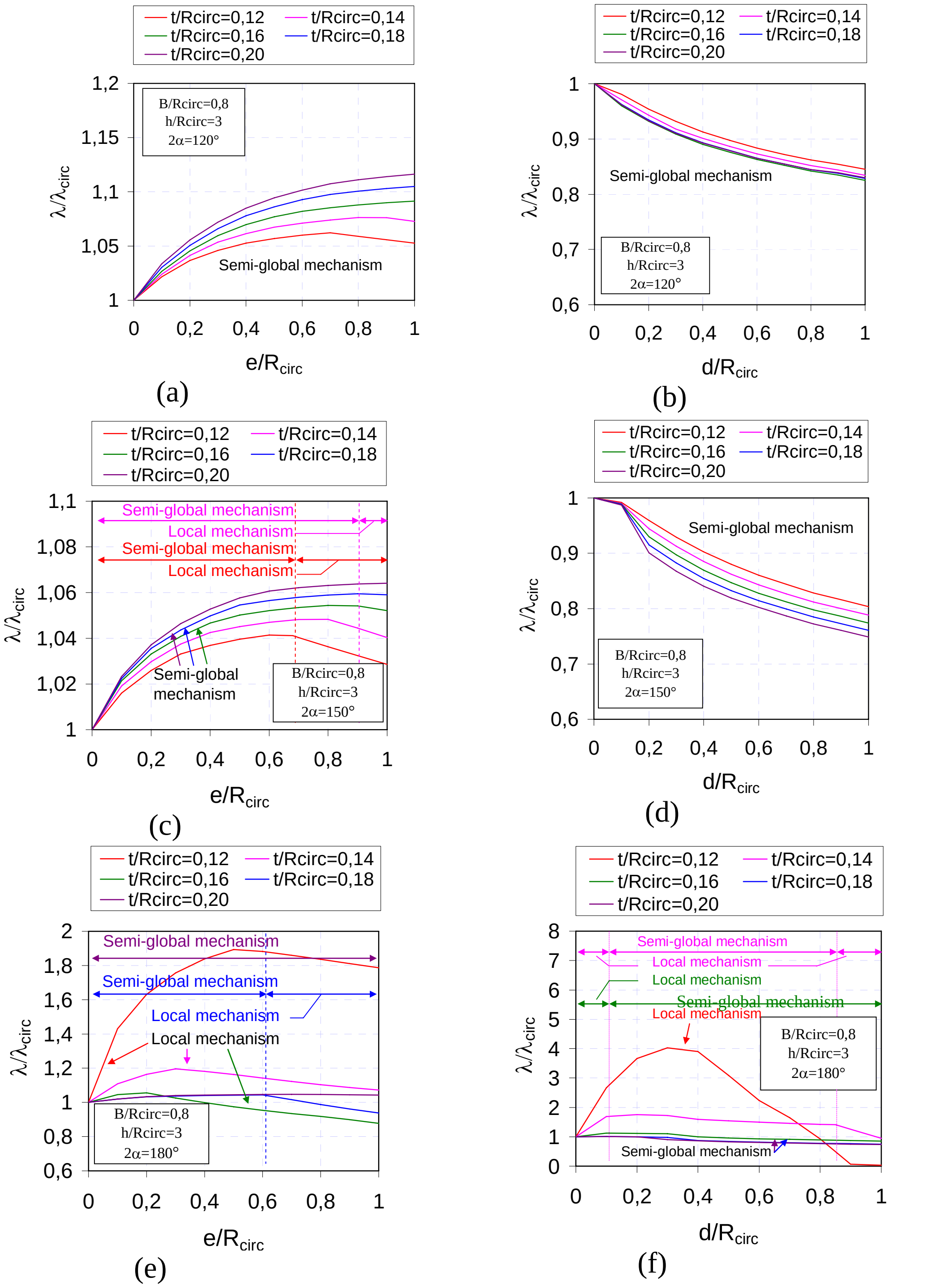


Figure 5 – Lateral stability of portals with pointed (a-c-e) and basket-handle (b-d-f) arches with angles of embrace 2 =120°, 150° and 180°

1. Pointed arches with angle of embrace equal to 120° and 150° are always less stable than the circular ones, and their stability decreases as the *e/R_{circ}* ratio (i.e. the degree of "pointedness") increases.

2. For an angle of embrace of 180°, thin pointed arches may be much more stable than their reference circular arches. For each thickness ratio, the collapse acceleration is maximum for a certain e/R_{circ} (optimal ratio).
3. Basket-handle arches with angle of embrace equal to 120° and 150° are always more stable than the circular ones. Their stability in most cases increases as the d/R_{circ} ratio increases.
4. For 2 = 180°, thin basket-handle arches may be much more stable than their circular counterparts for moderate d/R_{circ} values, but become less stable for large d/R_{circ} values. For each given thickness ratio, the collapse acceleration reaches the maximum for a certain d/R_{circ} ratio (optimal ratio).
5. The collapse lateral acceleration of a generally shaped arch increases as the thickness increases and the angle of embrace decreases.
6. At collapse, a generally shaped masonry arch requires two hinges to form at the intrados and two at the extrados, with one hinge forming *always* at the extrados side of the support.
7. Portals with pointed arches are in most cases more resistant to lateral accelerations than those with circular arches. This holds both when semi-global and local collapse mechanisms control.
8. As the angle of embrace increases, the local mechanism controls for a wider range of e/R_{circ} values, especially for small thickness ratios.
9. Portals with basket-handle arches having 2 = 120° and 150° are always less stable than their circular counterparts, and the stability decreases as the d/R_{circ} ratio increases. The semi-global mechanism always controls
10. For 2 = 180°, the stability may become much larger than that of the portal with circular arch for thickness ratios between 0.12 and 0.14 and “moderate” d/R_{circ} values.

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PRELIMINARY STRUCTURAL ASSESSMENT OF KARIYE MONUMENT - NORTHERN ANNEX

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ABSTRACT

In this study, the architectural and structural features of Kariye Monument are outlined, with particular emphasis on the evaluation of structural behavior of Northern Annex. The evaluation study included extensive on-site observations, preparation of in-situ drawings of the existing structural system, damage assessment, and two and three dimensional structural analysis of the Northern Annex, as well as examination of existing documents on Kariye Monument. The observed damages in the Northern Annex are found to be consistent with the structural analysis results, which included the effects of foundation settlement and rotation as well as vertical loads.

1. INTRODUCTION

Kariye Monument, which is among few remaining examples of Byzantine period, is briefly described in various books and articles on Byzantine architecture [3], [5]. These documents represent Kariye Monument as a unique example of the 14th century Byzantine art. The building was studied in depth for the first time during the restoration works of American Byzantine Institute (1947-58). Except for some architectural/structural interventions especially in the eastern parts of the monument, the works undertaken during these years focused mainly on the mosaic and fresco restoration. However, the excavations, photographs, plans, sections, elevation drawings and the observations of this period are of great value and constitute the foundations of the later studies. The summary reports of the Kariye excavations were published in 1960 [6]. Three volumes published by P.A.Underwood in 1966 were also based on the findings of the same period;

however, except the introduction of the first volume, Underwood also focused on the mosaic and fresco art of Kariye monument [10]. Almost two decades later, R. G. Ousterhout published in 1987 the only detailed and in depth study that deals with the architectural and structural features of the monument. His study included valuable insights into the structural weaknesses of the monument [8].

In this study, after an outline of the history and architectural characteristics of Kariye Monument as well as some construction features, the observed damages and structural analysis results are presented for the Northern Annex of the Monument. The two dimensional and three dimensional analyses were in good agreement with each other, as well as the observed damages of the vaults of the Northern Annex.

2. HISTORY OF KARIYE MONUMENT

Kariye monument, located in Edirnekapı, Istanbul, is very close to Theodosian Walls that constitute the border of Historic Peninsula today and the border of Byzantine Constantinople. Although the history of the monument dates back to 6th century, no reliable information exists for the period until 11th century. In 11th and 12th centuries, extensive rebuilding and enlargements were carried out. At this period, the monument was transformed into a church or small monastic complex. During and after French and Italian invasions and occupations in the first half of the 13th century, Constantinople was plundered and burned down, and many churches were destroyed and robbed until 1261. Michael Palaeologos during his reign (1261-82) encouraged the reconstruction and repopulation of the capital [9]. When Kariye was rebuilt in the second decade of the 14th (1315-21) century, large churches were not being built anymore due to smaller religious community and economic situation. The attitude of the donors and builders during the Palaeologan Dynasty was to repair and enlarge the existing churches and monasteries rather than building new ones [3]. The donor and very probably the designer of the 14th century Kariye Church was Theodore Metochites, who had very close relationship with the Palaeologan family. He was also the person, who ordered the rich mosaic and frescoes in the monument. After the conquest of Constantinople by the Ottomans in 1453, the church was converted into a mosque (1511) [7]. The bell tower was immediately replaced with a minaret, mosaics were plastered. Some repairs were also made later, mainly to improve the building structurally or to keep the water away from the interior. There is no information about the damages caused by 1509 and 1754 earthquakes. In 1766 earthquake the main dome of the monument collapsed. It was replaced by a wooden dome during the restoration undertaken in the same year [5]. Half of the minaret fell down in the earthquake of 1894.

The monument was turned into a museum in 1945. The monument and its mosaics and frescoes were restored by the American Byzantine Institute during the Republican years between 1947 and 1958. Kariye Monument is famous for its mosaics and frescoes that are the unique examples of Byzantine Art.

3. ARCITECTURAL CHARACTERISTICS AND CONSTRUCTION

The building settles on an area of 27.5×27 meters. The main dome, which is carried by 12th century arches and piers measures 7.45 meters in diameter. Four arches that transfer the load of the dome to four corner piers rise approximately 10.35 meters above the floor level. Over the cornice, which ends at 11.52 meters, comes the dome that rises another 6.40 meters [8]. The dome is rebuilt from wood and plaster (bagdadi technique) in the Ottoman period in 1766, possibly after the earthquake in the same year [8]. Both prothesis and diaconicon that constitute the pastophoria are rectangular in plan. Those were turned to squares with the help of vaulting. The parecclesion, which functioned as a funerary chapel and sheltered the tombs of Theodore Metochites and other members of the aristocracy, was entirely constructed in the 14th century, including two long barrel-vaulted cisterns beneath the ground level. The second largest dome, which measures 4.70 meters in diameter, was situated in parecclesion. The irregularity of the rooms between naos and parecclesion show clearly the difficulty of adding new space to the existing naos. The northern annex, which was used as Metochites' library, has two floors. Lower floor is rectangular in plan measuring internally 10.14×2.99 m and is covered with barrel vault. Upper floor measures internally 2.97×9.79 m and is covered with barrel vault that rises 3.50 m above the floor.

The oldest foundations of Kariye date back to the 6th century (Figure: 1, phase 1) [8]. Two brick arches, which were part of an arcaded wall survived in the eastern part of the building. Ousterhout claimed that those were used as a funeral crypt, but the function of the building they belong is not known [8]. Those arches were blocked (strengthened) in the 9th century and were used as a terrace wall supporting the next building (Figure: 1, phase 2). The 11th century brick construction survived in the lower naos walls (Figure: 1, phase 3). In 1120, the church was rebuilt and the central space, dome and apse were enlarged. Except for the dome, which was rebuilt in the Ottoman period, the piers, walls, arches of the naos and bema, which survive today, were built in that time (Figure: 1, phase 4). The crusade armies' occupation (1204-1261), the earthquake of 1296 and the rioting of Catalan Company (1302) may have caused serious damages to the 12th century Kariye Monument. The church was repaired and enlarged by Theodore Metochites in the 14th (1315-1321) century; pastophoria, northern annex, parecclesion, two narthexes, the flying buttress (that supports the apse) and bell tower were constructed (Figure: 1).

The three layered walls of 11th and 12th centuries Kariye (naos and main apse) were built with the recessed brick technique [8]. This is common in almost all of the buildings in Constantinople constructed in the same period. Recessing every second course of brick roughened the inner surface and bonded firmly the rubble core of the wall (Figure: 2). The use of rubble core walls was common in Constantinople, because bricks were no longer available in the Middle Byzantine Period as they were in the earlier centuries [3]. The construction of the 14th century Kariye is almost entirely solid masonry with little or no rubble fill; this could be traced through the wall pattern, which is the same in the interior and exterior [8]. In Kariye Monument, the bricks are used alternating with stone with the pattern of four courses of brick to four of stone. This pattern changes only in prothesis and north annex, where three courses of brick alternating with four of stone (Figure: 3). Byzantine mortar includes lime, sand and crushed brick. The ratio between thickness of brick and mortar joint, which changes throughout centuries, is approximately 1:1.5 in 11th and 12th century walls, and 1:2 in 14th

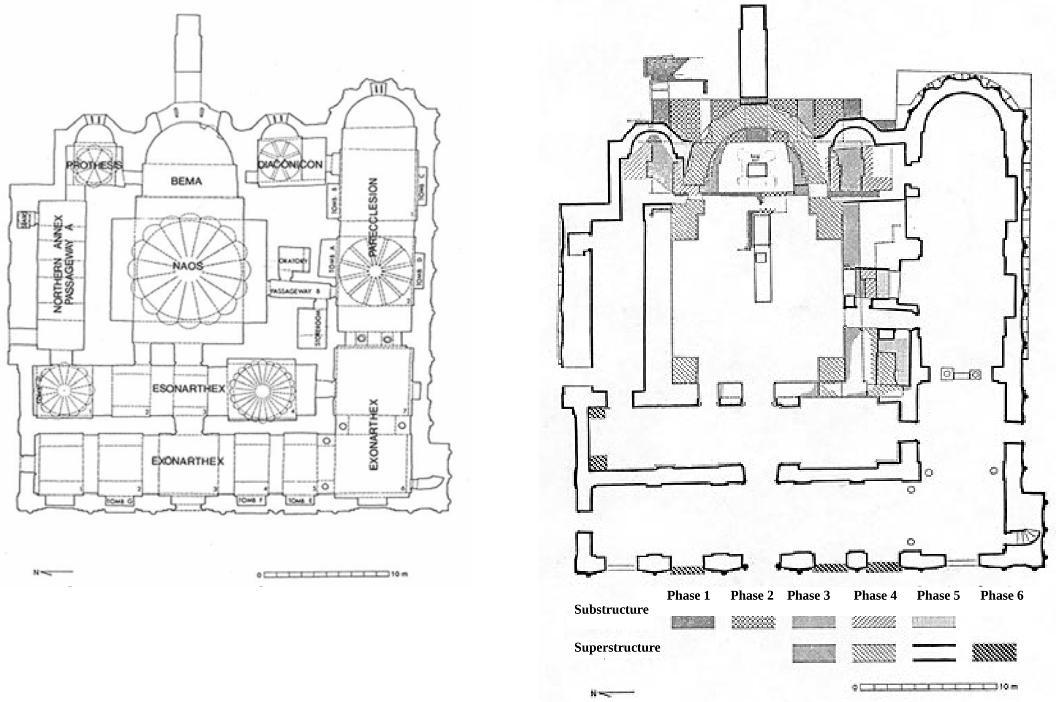

Figure 1 [8]

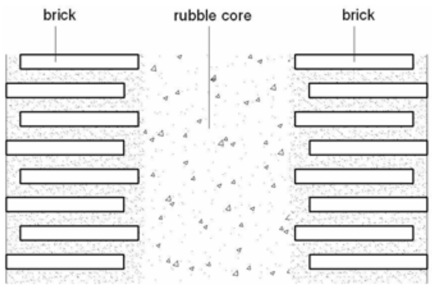

Figure 2 Figure 3

4. DAMAGES

The terrain of Kariye monument has been shifting downhill. This phenomena caused cracks in the Eastern part of the building. A 8 cm wide crack, which occurred in the eastern bay of parecclesion, was repaired using cement mortar during 1950s (Figure: 4). The shifting of the terrain was also the reason for the construction of a flying buttress in the 14th century to support the eastern wall of the apse and later the collapse of buttress's central flyer, which was restored in 1950s (Figure: 5).

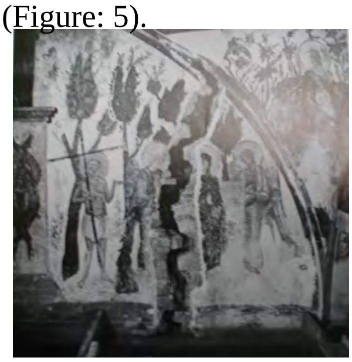

Figure 4 [8]

Figure 5 [8]

During the Ottoman restoration of the monument in 1875-76 to level the roof, a considerable amount of rubble was placed. This intervention not only changed the appearance of the monument, but also brought extra load to the structure. The appearances of the monument before 1875-76 restoration and today are presented in Figure: 6. The architectural additions made to the 12th century naos and bema had to be adapted to the older elements and meet certain functional requirements of the period, resulting in a structural irregularity, which is pointed out to be the major factor of the monument's structural weakness [3], [8]. According to Ousterhout, lack of vertical coordination of windows in the eastern wall of the north annex is the reason of the wall crack there [8]. The vertical non-alignment could be observed also in the western wall of the north annex, where the vent window is not aligned with the entrance door (Figure: 7).

Figure 6

Figure 7 Figure 8

Moreover the south wall of the north annex, built as a facing of the 11-12th century naos rests on the preexisting foundations, while the outer north wall rests on the 14th century foundations. The settling of northern foundation seems as one of the reasons for the cracking of the vault covering the lower chamber of northern annex (Figure: 8). The vault was strengthened by the Ottomans who added three pointed arches of brick and stone.

5. STRUCTURAL ANALYSIS OF THE NORTHERN ANNEX

In order to explain the causes of large cracks on the vault covering the lower chamber of the Northern Annex, a number of finite element analyses were performed on a portion of the structure. Two and three dimensional analyses were carried out using commercial finite element programs SAP2000 v.9.03 and ABAQUS v6.6-1, respectively [2], [1]. For the sake of simplicity and reduced run time, linear elastic and isotropic constitutive laws were utilized for material characteristics of the masonry components. The material characterizing the alternating brick-stone masonry walls and the brick vaults covering the lower and upper chambers of the Northern Annex were assumed to have an elastic modulus of 1000 MPa, poisson ratio of 0.15 and density of 2000 kg/m^3. These values are compatible with the values given for this type of structures [4]. The two dimensional simplified model was created using equivalent frame members, while 261716 four-noded C3D4 type tetrahedral elements with an average element size of 200 mm were used for three dimensional sophisticated analysis. The view of the Northern Annex and the finite element model are shown in Figure: 9.

The initial three dimensional finite element analysis was performed for the vertical loads, which were determined as the self-weight of the structure. Apart from the load bearing structural components, the weight of the infill material used for leveling of the upper chamber was also included in the analysis. Under the effect of self-weight, maximum compressive stress in the main walls of the lower parts of the structure was around 0.18 MPa, which is well below the compressive strength of brick and stone masonry. An average compressive stress of 0.10 MPa, and tensile stress of 0.10 MPa were obtained for the upper and lower faces of the mid-span of the lower story vault, respectively. From these values, it can be concluded that the intensity of the stresses induced by the self-weight of the structure is not sufficient alone to explain the damage pattern shown in Figure: 8. The deformed shape of the structure and the close-up view of the maximum principal stress distribution at the vault cross-section are given in Figure: 10. Following the statements done by Ousterhout [8] and the in-situ observations, it was decided to investigate the effect of settlement of the ground. After a number of trials on the type and the amount of a probable ground settlement, it was seen that a differential settlement causing around of more than 6 mm difference between the opposite sides of the northern façade wall may cause the damage pattern shown in Figure: 8. The deformed shape of the structure and the close-up view of the maximum principal stress distribution at the vault cross-section obtained only for the differential settlement are given in Figure: 11. As seen in this figure, the maximum principal tensile stresses at the lower face of the mid-span and the upper portion of the vault-main wall connections are higher than the assumed 0.2 MPa tensile strength of the masonry. As seen in Figure: 11, the locations of tensile stresses higher than the tensile capacity of the vault correspond to the actual crack locations observed on-site.

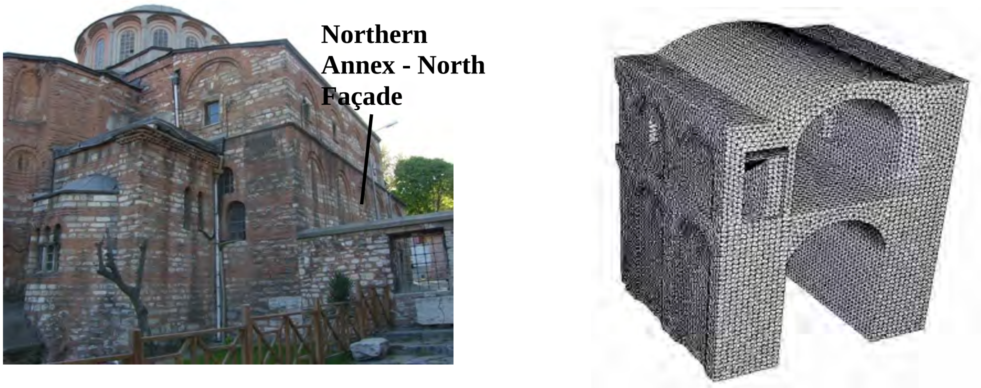

Figure 9

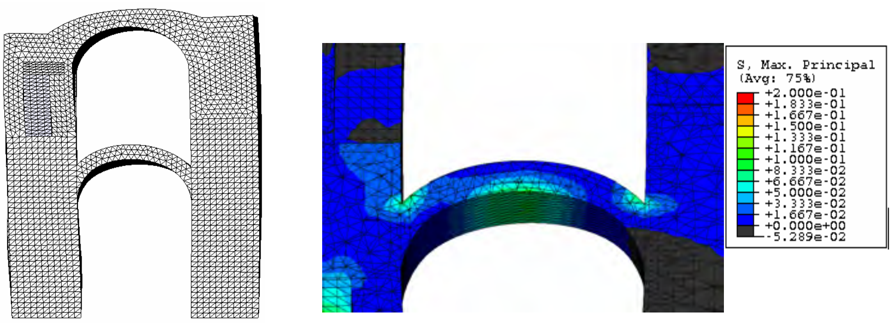

Figure 10

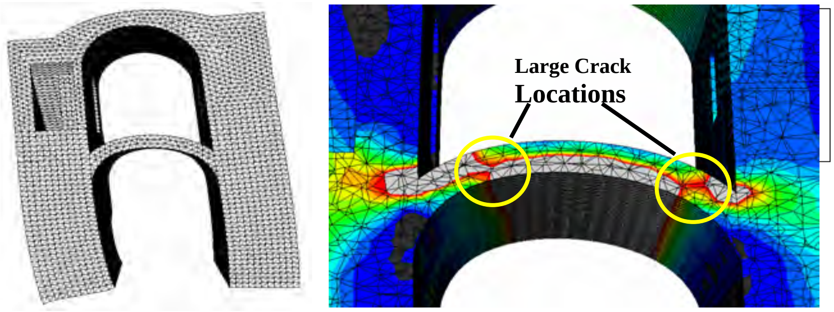

Figure 11

The maximum principal tensile stresses formed due to combination of self-weight and differential ground settlement explained above are very close to stresses formed due to ground settlement only. This indicates the significant contribution of differential settlement to the overall behavior

6. CONCLUSIONS

According to structural analyses carried out using FEM and on-site damage observations, it is concluded that the damages in the vaults of the Northern Annex are not formed due to the self weight of the structure, but formed as the result of differential settlement of the foundation of the north wall of the Northern Annex is thought to settle on relatively shallow 14^{th} century foundations.

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THE INVESTIGATION OF THE DYNAMIC BEHAVIOR OF HISTORICAL MASONRY MINARETS

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ABSTRACT

In this study, the dynamic behavior of the old masonry minarets, which usually exhibit vulnerable behavior under seismic and wind loads is investigated. The mechanical properties of the masonry material and installation and the analyzing techniques used in this field are investigated. Then, the application of computational analyzing techniques is discussed. Next then, in-situ studies performed on the minarets are studied. The results of micro tremor measurements are presented. Finally, a finite element model is proposed for the minaret using shell elements with SAP2000 program. The study is concluded with the modal analysis results of the finite element model with the proposed mechanical properties of the structural system.

1. INTRODUCTION

As Historic buildings and monuments need to be structurally protected and restored. They are subjected to aging effect and some of them may also be affected by seismic action. Structural evaluation of such buildings has a special importance as being the very first step of engineering interventions. Masonry minarets are one of the most beautiful members of the Ottoman Architecture. Unfortunately, there is not enough study on the dynamic behavior of these historic heritages.

Today, it is a very well known fact that earthquakes are one of the most important problems that Turkey has to tackle with and being established on the worlds oldest landscape brings the problem of seismic protection of historical heritages with it. It has been always challenging for engineers to analyze and design these buildings due to highly complex behavior of the material used in the construction. The problem becomes more complex when the dynamic analyses are also involved. However, after the developments in the dynamic testing of structures and computational methods in structural analysis, studies in this field have led some significant results about the mechanical behavior of the old buildings. These types of studies are essential for not only in protection point of view but also assessment of ground motion of the past events.

Within this frame work, the dynamic behavior of old masonry minarets, which usually exhibit vulnerable behavior under seismic and wind loads is investigated. In this study, the mechanical properties of masonry material and the analyzing techniques used in this field are presented then the application of computational analyzing techniques is discussed.

The study is based on a case study on the Dolmabahçe Mosque's minarets, which is conducted by a research team from Department of Civil Engineering at Istanbul Technical University. This chapter explains the in-situ studies performed on the minarets. A brief description of the structure and the results of micro tremor measurements are presented in this chapter. Then a finite element model is proposed for the minaret using shell elements with computer program. Also, the shell element and modal analysis in computer program is summarized briefly. The study is concluded with the modal analysis results of the finite element model with the proposed mechanical properties of the structural system.

2. MASONARY MINARETS AND THEIR STRUCTURAL PROPERTIES

Masonry minarets are one of the most significant architectural objects of the cultural inheritance from the period of Ottoman Empire. They were considered as a sign of the Empire's reign supreme on its territories and also a symbol of Islamic Culture all over the world. In order to assess the seismic vulnerability of the old minarets, it is necessary to know the construction technique of them. The observations about the geometric characteristics of minarets and their special construction technique can be observed from reconstruction facilities.

In general, a minaret consists of three main parts as it is shown in Figure 1. The inner diameter of the minaret is constant along the full length while the thickness of the wall is changing. The footing is constructed out of very thick stone blocks and firmly connected with the wall of the mosque with which it forms unique compact structure, while the higher part of the minaret above the mosque walls, presents slender cantilever structure [1].

The lower part, from bottom to the gallery, is constituted by the wall, the stairs and, the core. The thickness of the wall in this part decreases along the length. Inside of the upper part, from gallery to the top of minaret, is empty. The thickness of the wall and the outer radius of this part are constant along the length. The dimensions of this part are smaller than the lower part's.

As it is mentioned in the first chapter, the tensile strength of masonry material is considered to be zero. This is the major problem of masonry structures, which are subjected to big lateral loads during earthquakes, especially the slender structures like minarets and bell towers. In the beginning of 16th century, after the major earthquake in 1509, Ottomans tackled the problem of building earthquake resistant, higher minarets. They started to use a special type of reinforcing method, which is basically linking the stone blocks with forged iron connectors in vertical and horizontal directions, instead of using traditional mortar. Apparently the purpose of using reinforcement was to resist the high tensile stresses that will occur due to lateral loads, i.e., wind loads and earthquake loads [2]. The use of lead as a filler to anchor the connectors into stone is the other main characteristic of this unique technique.

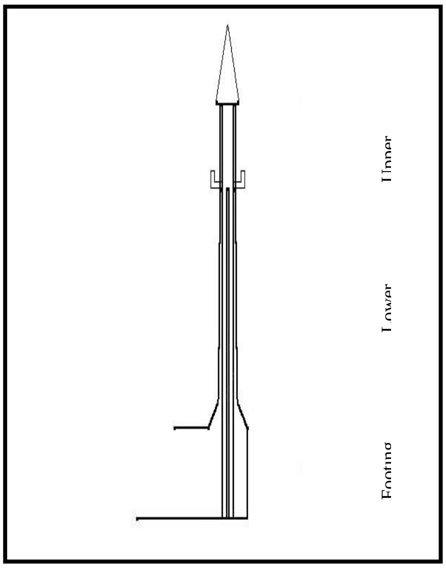

Figure 1. Main parts of a minaret

3. INSTALLATION OF STONE BLOCKS FOR CONSTRUCTION OF MINARETS

The installation method of stone blocks makes this technique exclusive. This process is briefly, connecting each block to six adjacent blocks by two clamps and four bars. The positions of these connectors are given in Figure 2. Anchorage holes are 4 x 4 x 6 cm rectangular prisms (6 cm is the depth) and opened at site by workers. The locations of the holes are inconsistent, because workers adapt their places during construction, according to adjacent blocks positions. Nevertheless, the clamp anchorages holes are in the vicinity of outer corners of the stone blocks where bar holes are close to the inner faces of the blocks. The installation steps, observed during the reconstruction of Fındıklı Mosque which is common can be described as below.

Step1: Two bars are placed in the holes and fixed firmly with lead filler on the face that is going to be the bottom face of the stone block after installation.

Step 2: The stone block is turned upside down and mounted on the wall, such that the bars fit in the holes whose locations were adapted previously by workers on the lower adjacent blocks surfaces. These holes are filled with melted lead from their openings to inner face of the wall.
Step 3: The first and the second steps are repeated for each stone block in the ring. Once all blocks in the ring are installed, they are connected to each other by clamps, which are also fixed in the holes by lead filler. The thicknesses of the clamps are embedded in the blocks in order to provide a smooth interface between blocks.
Step4: Finally, cement mortar is injected into the gaps between the stones and construction of a ring is completed. The step stones are connected with the wall in the same manner and they are also connected with each other by bars from their core parts.
Consequently, minaret is erected by repeating these steps at each ring level. It is evident that this procedure is very time consuming, so that only 8 or 9 blocks can be installed in a day.

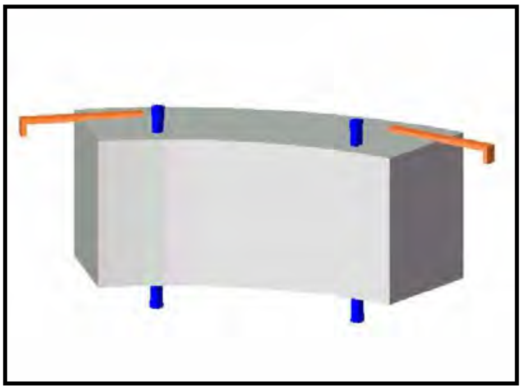

Figure 2. Positions of connectors on a stone block

4. THE STRUCTURAL MODEL OF THE MINARET

In order to determine the equivalent elasticity modulus of the reinforced masonry system; modal analysis was performed using Finite Element Method, on the same minaret that is tested in-situ before. The model is created according to the related dimensions of the minaret of the Dolmabahce Mosque. The analysis is performed by SAP 2000 [3].

Most of the dimensions of the minaret of the Dolmabahce Mosque have been measured manually. The height of the lower parts of the minaret, from the bottom to the gallery, was obtained by measuring each step stone’s height. The change in the thickness of the minaret’s wall was estimated by measuring the thickness of the air holes, which are located at every 3~3.5 meters on the southeastern side of the minaret. However, it was not possible to reach to upper part of the minaret, because there is no stairs in this part. In order to determine the dimensions of this part, photogrammetric measurement is made.

In order to provide the consistency in appearance, the Maktarali Limestone has been used in the construction of all buildings in Dolmabahce Complex. During the restoration of the Dolmabahce Palace, numerous experiments are done on the stone specimens obtained from the debris of the old structure. The Department of Mining At Istanbul Technical University conducted this study and the final report [4] is published in March 2000. In this report, mechanical properties of Maktarali Limestone are given in Table 1. The foot of the minaret is modeled as an octagon prism rather than a cylinder. The wall of the minaret was modeled using quadrilateral (four node) shell elements. The variation in the wall thickness (30 cm to 21 cm) of the lower part is simulated by decreasing the thickness of the shell elements by 1 cm at every 2 meters. The steps were modeled as an assembly of five triangular shell elements with 24 degree subtended angle. The graphical output of the structural model of the minaret is given in Figure 3.

Table 1. Physical properties of Maktaralı Limestone

Physical Properties	Max	Min	Average
Dry Density (t/m^3)	2.49	2.30	2.39
Fully Saturated Density (t/m^3)	2.53	2.37	2.45
1D Compression Strength (Mpa)	19.27	12.30	16.78
Tensile Strength (Mpa)	0.95	0.88	0.90
Young's Modulus (Gpa)	7.36	4.30	5.84

The same density of the Maktarali Limestone given in section is used in the model (ρ=2.39 t/m^3). The masses of the gallery and the top (kulah) of the minaret are introduced as lumped masses. The Poisson's ratio (ν) is based on past studies, in which it is assumed to be 0.24 for a good quality of masonry. [5].

The Shell element is used in the model as a three or four node formulation that combines separate membrane and plate-bending behavior. The four joint elements do not have to be planar. Each Shell element has its own local coordinate system for defining material properties and loads, and for interpreting output [6, 7].

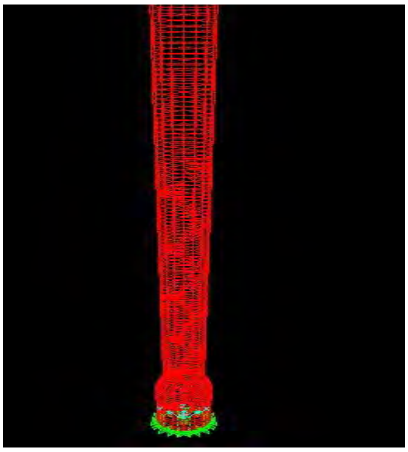

Figure 3. The structural model of the minaret

5. THE ANALYSIS AND RESULTS

The natural modes provide an excellent insight into the behavior of the structure. They can also be used as the basis for response-spectrum analyses. Eigenvector analysis involves the solution of the generalized eigenvalue problem:

$$[K - \Omega^2 M]\ \Phi = 0 \tag{1}$$

Where K is the stiffness matrix, M is the diagonal mass matrix, Ω^2 is the diagonal matrix of eigenvalues, and Φ is the matrix of corresponding eigenvectors (mode shapes). Each eigenvalue eigenvector pair is called a natural vibration mode of the structure. The Modes are identified by numbers from 1 to *n* in the order in which the modes are found by the program. The eigenvalue is the square of the circular frequency, ω, for that mode. The cyclic frequency, f, and period, T, of the mode are related to ω by:

$$t = 1/f \tag{2}$$

$$f = \omega / 2\pi \tag{3}$$

A mass degree of freedom is any active degree of freedom that possesses translational mass or rotational mass moment of inertia. The mass may have been assigned directly to the joint or may come from connected elements. The main mode shapes of the minaret are given in Figure 4 which obtained from the FEM analysis. The sensors were placed on the gallery and in the second air hole from the bottom of the minaret for micro tremor measurement. Totally 12 time-series were recorded in horizontal direction. The processing of data consisted on the computation of the Fourier spectrum for each time series. The frequency of the first natural mode is obtained by averaging the Fourier spectrum for the whole series.

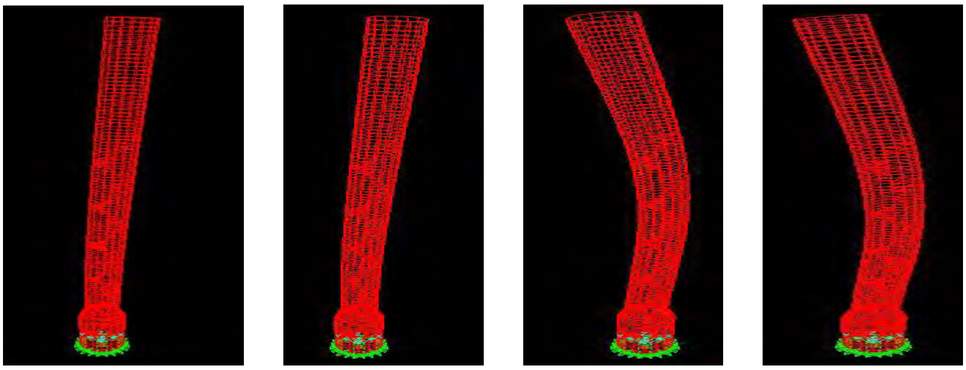

Figure 4. The main mode shapes of the analyzed minaret

The first natural frequency obtained from these calculations is 0.87 Hz, i.e. the first period of the structure is 1.15 sec. This value also can be easily observed on the response spectrum presented in Figure 6, which is extracted from Figure 5 for a particular time range. The differences between the occurrences of peak displacements are about 1.15 sec.

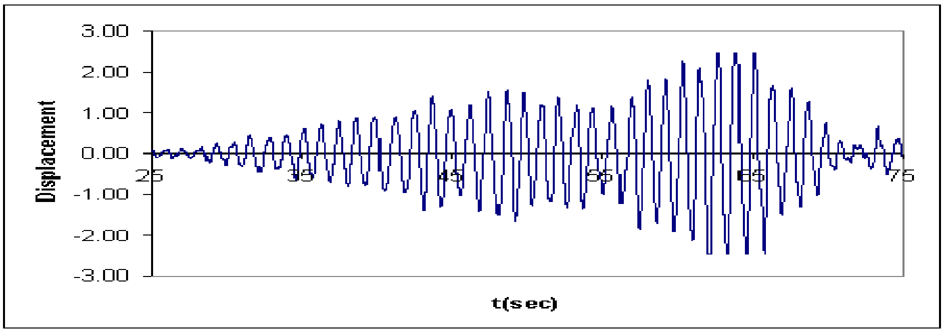

Figure 5. Sample Fourier spectra on N-S direction

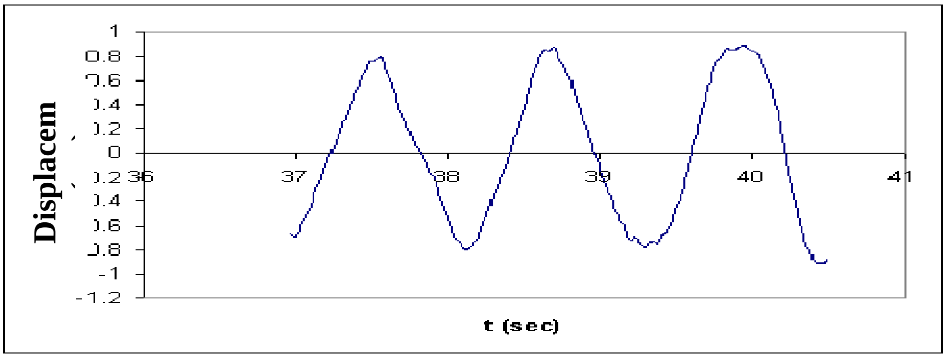

Figure 6. Sample Fourier spectra on N-S direction

6. CONCLUSIONS

The large intensity earthquakes are the most important danger for historical monumental buildings. That's why the dynamic response of this kind of structures must be studied. However this is very difficult and complicated works because of the complexity of the problem. Especially masonry minarets have a special place in this topic. It has been always challenging for engineers to analyze and understand the response of these buildings due to highly complex behavior of the material used in the construction. The problem becomes more complex when the dynamic analyses are also involved. A finite element model is proposed for the minaret using shell elements with computer program. The equivalent modulus of elasticity obtained to fit the result of experimental testing was E = 72 Gpa, this value yields to 1.15 sec for the first natural period of the computer model. That value is obviously above the values one could anticipate, but it also reflects that the minaret is in presence of a very well constructed structure and the construction technique developed through centuries.

The material properties and force displacement behavior has a key role to understand the dynamic response of minarets. In order to develop better analysis alternatives, it is necessary to develop better models for material and force displacement behavior which reflects better the real case.

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STRUCTURAL MODELING, EARTHQUAKE ANALYSIS, AND STRENGTHENING PROPOSALS FOR GAZIANTEP ALI NACAR MOSQUE MINARET

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ABSTRACT

Gaziantep Central Ali Nacar Mosque was constructed around the 15th century, which has a minaret of about 2.6 m in diameter and 20 in height. The mosque and its minaret is important from a historical point of view; however, the minaret is visibly inclined towards east with about 1.5 degrees and shows signs of structural problems such as deep cracks in the stone blocks. The tilt of the minaret is thought to be due to foundation problems and progress of the tilting is uncertain for the time being. Authorities are concerned with the structural condition of the minaret as well as its structural performance during a possible earthquake. This paper presents the structural modeling and earthquake analysis of the minaret, along with proposed methods for structural strengthening.

1. INTRODUCTION

The approach to the structural evaluation involves 3D analytical modeling and simulation studies. The effect of tilt on the structural stability is evaluated using the tilted analytical model. Additional response spectrum analysis was conducted to obtain stresses developing in the minaret. Development of tensile stresses in the body of the minaret was accepted as signs of brittle failure and related damage mechanisms. Strengthening methods were proposed in an attempt to improve current structural condition, since minaret's currently tilted position with existing cracks and weak binding between stone blocks may not be safe to successfully carry the dead load and earthquake forces. Modeling, simulation, evaluation studies, and strengthening suggestions including foundation remedies are discussed in detail under each heading below.

2. ANALYTICAL MODELING

The structural model of the minaret was composed of 8 noded solid members due to the thick stone walls of the structure. The staircase and linking central column were also modeled using solid members. The door and window openings were incorporated into the structural model. Material characteristics were determined by using 8 × 8 × 8 cm sized cube specimens according to TS 699. Dry and wet specific weights of the material were found to be 17.3 kN/m^3 and 20.2 kN/m^3, respectively; and the mass was accepted to be equal to 1800 kg/m^3. The ultimate compressive capacity of the material was also reported to be equal to 10 MPa, from which the tensile capacity was assumed to be 1 MPa. On the other hand, the tensile capacity may be conservatively accepted as zero from a practical point of view, assuming that there are no mechanical or chemical connection between the stone blocks as well as stress concentrations are already too high at the wedge of existing cracks. The elastic modulus of the material is not available, but assumed to be approximately equal to 10,000 MPa based on the compressive strength of the material.

The constructed analytical model has about 2400 solid members and about 11 000 degrees of freedom. The solid members were defined as shell members in the construction process at the base of the tower as shown in Figure 1(a). The connection between the central column and the stair blocks were properly modeled using rectangular and triangular shell elements. The shell members were replicated in the vertical direction in a spiral pattern to form the bases of stairs. The shell members were extruded to convert them into solid members (Figure 1(b)) as the initial shell elements were deleted from the model. The stairs had touching corners (Figure 1(c)) as well as proper connection to the central core column (Figure 1(d)). The outer solid member was replicated in the circumferential and vertical directions to form the outer wall. Total of 14 solid blocks were used in the circumferential direction to form the minaret walls (Figure 1(d)).

The top section of the minaret is formed by a solid thick dome and carried by six columns (Figure 2). The roof is composed of wooden parts which were not considered to be structural parts and ignored for the analytical model. The columns were formed by using frame elements with the circular cross section having four pieces along the length. The compatibility of columns with the solid members is a critical problem. The frame elements have six degrees of freedom (dof) at the ends of the member while the solid members have only three dof for the translational direction motions. Since the bending moments in the frame elements cannot be transferred to the solid members at the nodes connecting the two members, the frame elements behave as if there are moment releases at the connecting nodes. The hinge mechanism formed by the lack of rotational dof in the solid elements can be corrected by using rigid links connecting the frame element end node to the nearby nodes in two orthogonal directions to the axis of frame member (Figure 2).

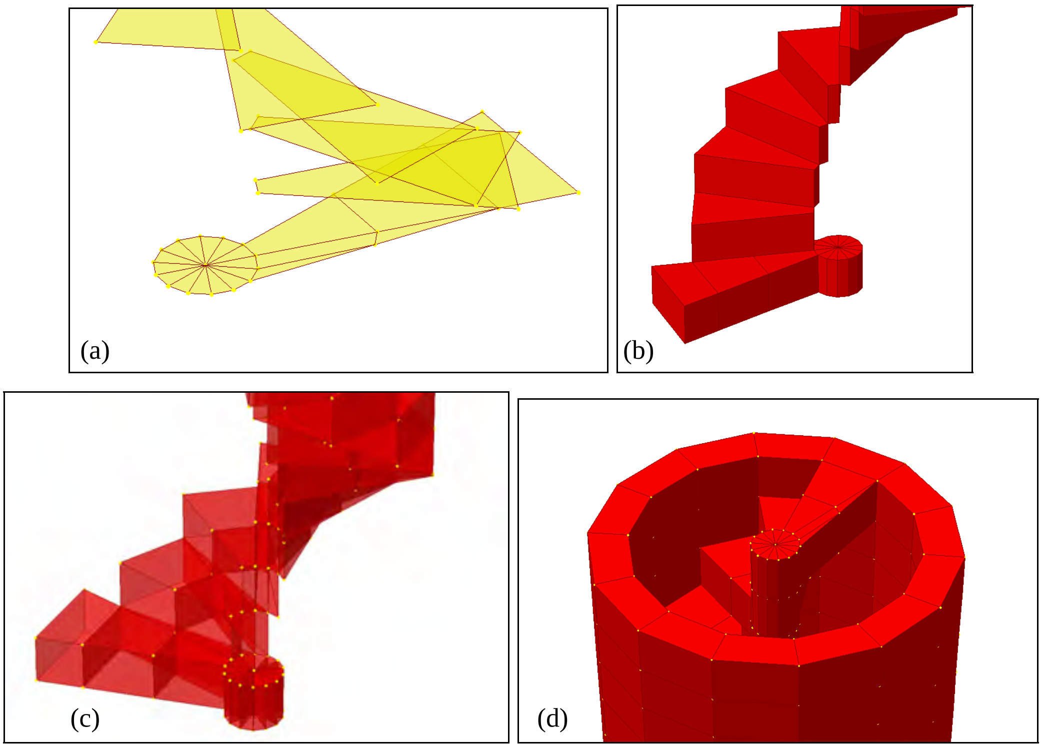

Figure 1: Analytical model generation of the minaret.

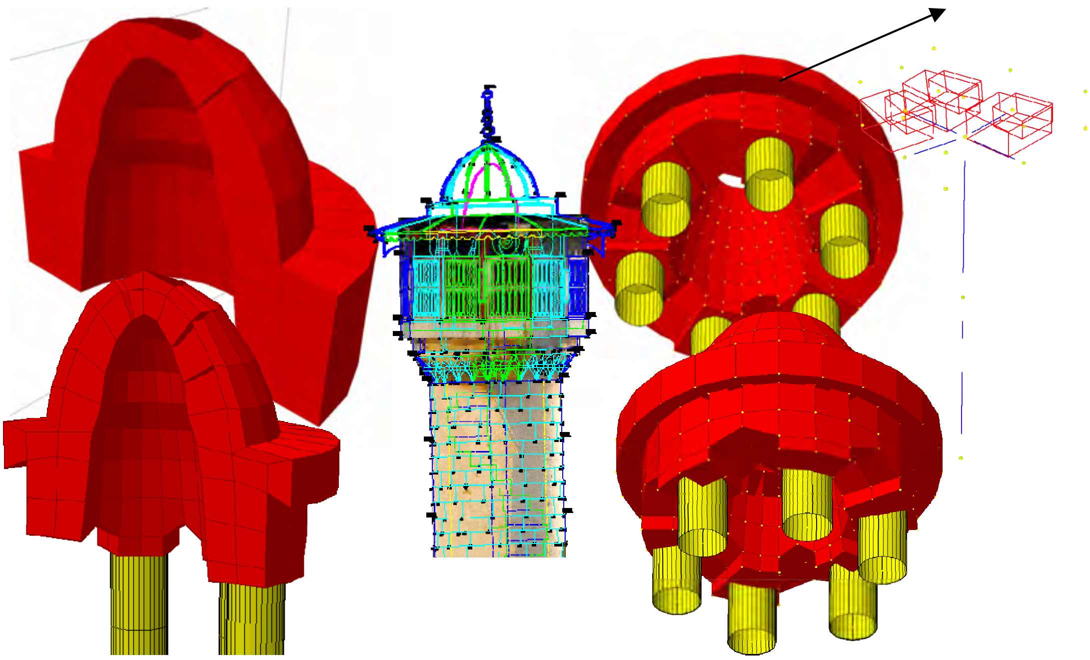

Figure 2: Dome supported by six columns at the top of the minaret.

3. DEAD LOAD AND EARTHQUAKE ANALYSES

The dead load analysis is conducted for the vertical and tilted conditions of the minaret. The stress concentrations at the top and bottom of the door opening in the wall are in agreement with the existing cracks at the top of the wall (Figure 3). Furthermore, stress concentrations in the middle of the stairs indicate possible locations of crack formations. The stone interfaces in the vertical direction and wedge of the existing cracks are susceptible to crack formation and propagation, although the tensile stresses are low in the range of 0.1 MPa on average and 0.25 MPa at the stress concentrations (Figure 3). Inclination of the minaret causes some insignificant level of increase in the calculated tensile stresses under dead load and the difference is small enough to be ignored. Existing cracks underneath the stairs support structural analysis calculations.

The tilt of the minaret is thought to be due to foundation – soil interaction since the minaret has a symmetric geometry. Considering more than 500 years of service life, the base rotation might be due to recent changes in the ground water levels such as draught or expanding soil due to a nearby water leakage from a pipe.

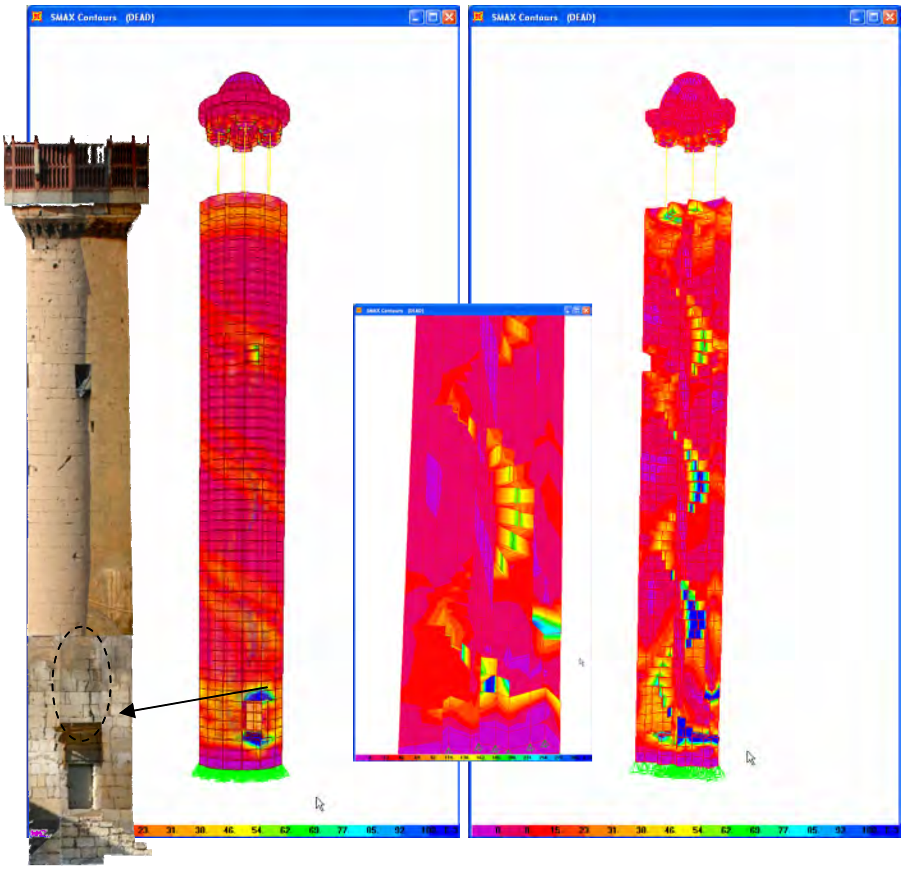

Figure 3: Dead load analysis results indicating stress concentration locations.

The earthquake analysis was conducted using response spectrum approach. The minaret is located on earthquake Zone 3 based on Turkish Earthquake Zoning Map, where Zone 1 is the most seismically active and risky region; therefore, ground spectral acceleration coefficient was taken as 0.2. The soil conditions were determined to be silty clay material (Type C according to Turkish specification for structures to be built in disaster areas, Part III - Earthquake disaster prevention), which refers to ground type Z2 having characteristic spectrum periods of T_A=0.15 sec and T_B=0.40 sec. The structural analysis of the minaret indicated that the first natural vibration period starts from 0.41 second and higher modes have shorter periods. Therefore, considering a Z3 type soil with T_A=0.15 sec and T_B=0.6 sec has no immediate effects on the linear response spectrum earthquake analysis results. The first six mode shapes obtained from the modal analysis are the two bending modes in x and y directions followed by torsional modes (Figure 4). The corresponding periods and frequencies of the modes are listed in Table 1. Although the first 30 modes were used for the earthquake analysis, only a limited number of the modes are presented here due to space limitations.

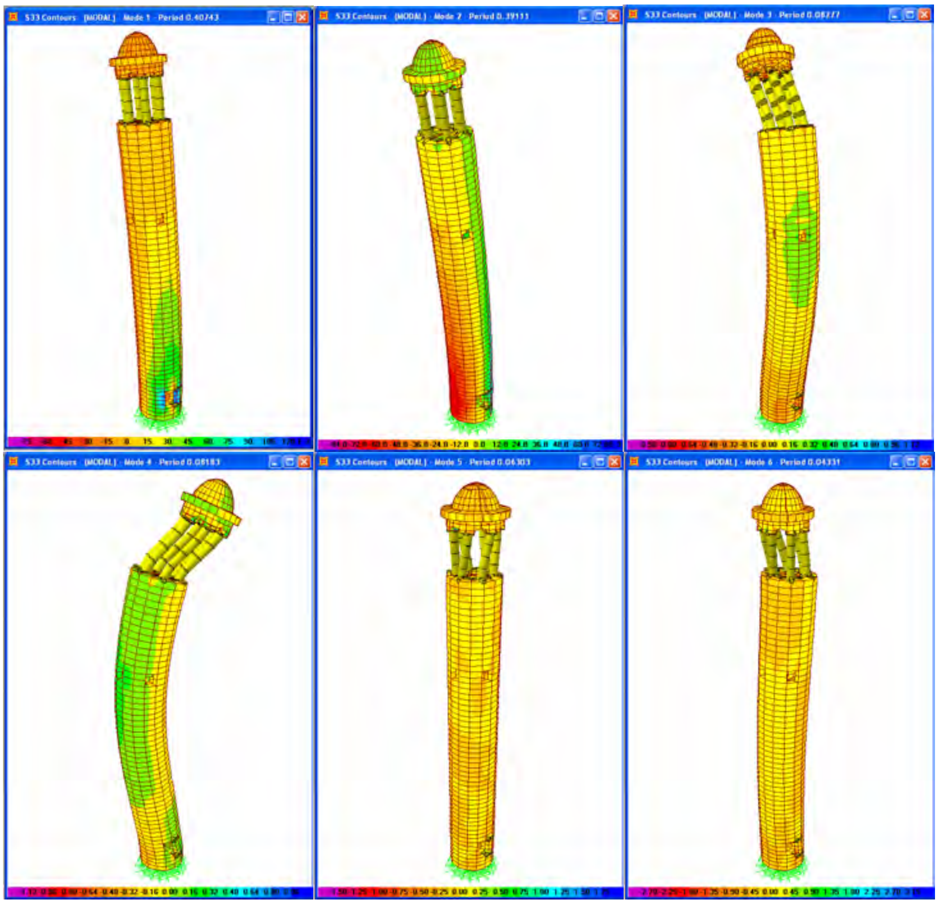

Figure 4: First six vibration mode shapes of the minaret.

Table 2: Calculated Modal Periods and Frequencies

Mode #	Period (sec)	Frequency (Hz)
1	0.407	2.45
2	0.391	2.56
3	0.083	12.1
4	0.082	12.2
5	0.063	15.9
6	0.043	23.1
7	0.042	23.7
8	0.042	24.1
9	0.035	28.7
10	0.022	45.9

The earthquake response spectrum analysis results show that tensile stresses are in the range of 3 MPa to 5 MPa developing at the base of the minaret in x and y directions (Figure 5). The level of vertical compressive stresses at the same locations change between 0.3 MPa and 0.7 MPa which are too low to counterweight the tensile stresses generated by the earthquake forces. The superimposed dead load and earthquake analysis yields tensile stresses in the range of 4 MPa which are beyond the capacity of stone blocks and connections between blocks.

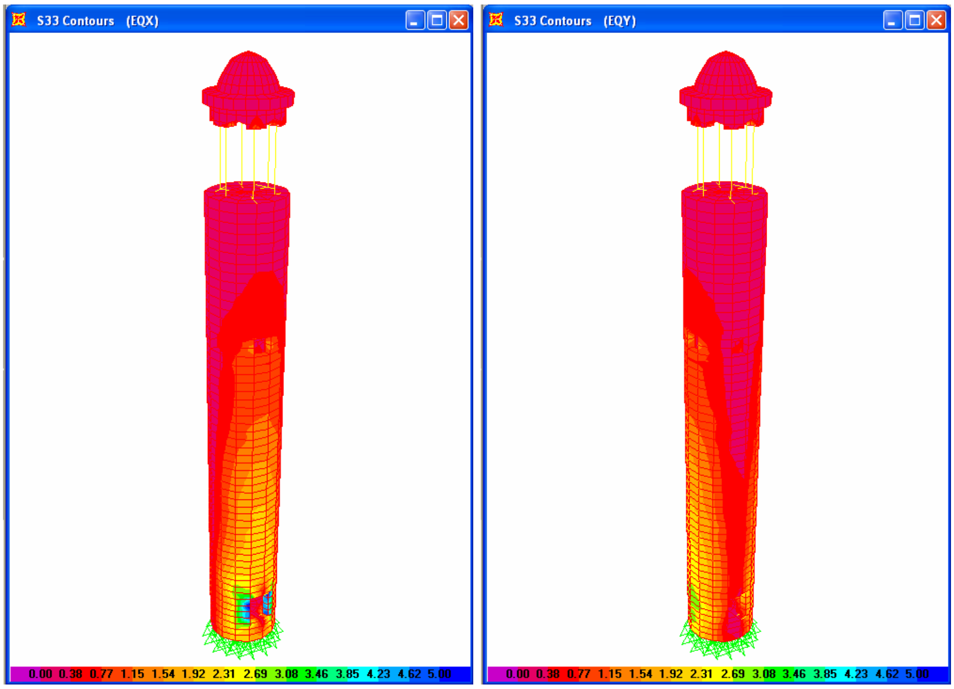

Figure 5: EQ analysis results showing vertical stresses.

The findings indicate that the minaret would not resist earthquake forces described in the currently available Turkish Codes and strengthening is necessary to improve its seismic performance. If the tilting of the minaret continues, at about 2.7 degrees of inclination (two times the current value), the existing cracks are expected to get larger and structure might get unstable as the maximum tensile stresses reach to 1 MPa accepted tensile capacity level. If the structural integrity is preserved, a rigid body overturning mode might be critical at 1.3 m lateral deflection at mid height of the 2.6m diameter and 20m tall minaret (at about 7.4 degrees of tilt angle, 5 times the current value).

4. STRENGTHENING SUGGESTIONS

Proposed structural strengthening methods include cross bracing at the top level, confinement of minaret walls using horizontally placed CFRP in circumferential direction, vertical direction post-tensioning using stainless steel rods, and application of vertical CFRP from the outer surface. The proposed methods should not be applied at the same time, but one or more methods may be applied concurrently. Specific details and working mechanisms are described below in further detail.

The small dome placed at the top of the minaret is supported by six columns, which are designed only for vertical (gravitational) loads. The columns would form a lateral failure mechanism during an earthquake. Cross ties are recommended to be placed between the opposing columns to generate lateral load carrying mechanisms.

The minaret walls have cracks in the vertical direction due to peripheral tensile stresses generating at the edge of the openings. Confinement using CFRP or stainless steel belts in the transverse direction would prevent propagation of cracks and provide additional shear capacity for the minaret; however, would not be a remedy for large tensile stresses developing in the vertical direction during a possible earthquake.

Vertical direction post tensioning would generate an even compression field in the vertical direction which would eliminate or minimize the tensile stresses generated by earthquakes. Post-tensioning can be from the outside of the walls, or by drilling holes through the walls if technological accuracy of the drilling tools permits such an operation. Alternatively, stainless steel may be replaced by vertical CFRP strips epoxied on the surface and locked at the top and footing level. In both cases, the vertically placed tension material has to be extended to the footing level and anchored in place at both ends.

The tilt of the minaret may be corrected by application of vertical pressure on the opposite side of the tilting, in an attempt to compress and plastically deform the soil on the lifting side to reverse the tilting action. Massive blocks of steel may be placed or a rigid platform can be pushed down utilizing hydraulic jacks connected to vertical bars anchored at about 30-40 m depth.

5. CONCLUSIONS

The structural condition and earthquake vulnerability of historical Ali Nacar Mosque's minaret was studied using a 3D-FE model utilizing 3D solid and frame elements. The dead load analysis of the minaret for the straight and tilted conditions showed that the tilting generates minimal level of additional stresses, since the tensile stresses generated by the dead load itself over the door opening is already too high. The crack pattern observed over the door opening and stairs correlate well with the linear structural analysis results. The existing 1.35 degree tilting angle would be considered critical at 2.7 degrees when the tensile stresses reach at 1 MPa level, which is the accepted critical tensile capacity of the material. The minaret would be structurally unstable and experience rigid body overturning at about 7.4 degrees of tilt where the center of mass would shift outside the base area. The small dome supported by 6 columns, each being 39 cm in diameter and about 2 meters in height, may become laterally unstable at about 5.8 degrees of tilt where the eccentricity in column loading would shift outside the column base areas.

Earthquake analysis using response spectrum defined in accordance with the Turkish seismic code indicates that the minaret is not safe during a possible earthquake. Although the minaret has a slender nature and relatively heavy due to its masonry construction, the calculated periods are smaller than 0.4 seconds, remaining in the linear and ascending part of the response spectrum. During an earthquake, the natural periods of the minaret are expected to get larger as structural damage progresses, which would move the periods towards the flat and maximum range of the response spectrum, causing the accelerations to get even larger. The minaret would likely experience heavy structural damage and collapse as the tensile stresses were found to be in the range of 4 MPa, developing at the base level.

A number of strengthening alternatives were suggested to improve the seismic performance of the minaret during a possible earthquake. The strengthening alternatives are currently in the form of mere ideas and should be carefully engineered according to minaret's structural response and incorporating the structural retrofit additions to the minaret. Controlled loading of the soil in the opposite direction of tilt might improve the condition.

Monitoring of the minaret's tilt is recommended regardless from the strengthening studies. Monthly manual tilt measurements or continuous instrumented monitoring would reveal the progress of tilting in the long run.

6. ACKNOWLEDGMENTS

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STATIC AND DYNAMIC ANALYSIS OF PANAGIA TON ISODION CHURCH

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ABSTRACT

Panagia Ton Isodion Church, one of the old churches of Istanbul and one of the two main churches in Beyoglu, is located near Cicek Passage in Galatasaray. The historic church is analyzed with a three-dimensional finite element model by using SAP2000 program to understand its structural behavior under the action of static and earthquake loadings. The structure, including all essential elements of the system, is modeled by using the solid finite element with 24 degrees of freedom. Structural behavior and the dynamic characteristics of church are demonstrated.

1. INTRODUCTION

Today, 94 churches, belonged to Fener Greek Orthodox Patriarch, survive in Istanbul as Temple. Panagia Ton Isodion Church, which is one of these churches, was constructed between June-September, 1804. Today, the building serves as a church and it is accepted as a typical example of 19th Century Byzantine Church Architecture [1-2]. It is not observed any deformations on the structure of the building although more than two centuries have passed over its construction. The church exhibits only some cracks on a small scale on the vaults, decay on the wooden piers, some breaking on the wooden floor, some rubbish on the walls and some trash on the plasters of the walls. It is required to analyze the structural behavior of the building under the static and earthquake loadings, and to repair and restore by Activity Navigation S.A. The aim of this research on the structural analysis of the Panagia Ton Isodion Church is to understand its structural behavior under the action of static and earthquake loadings. The structural behavior and the characteristics of its structural load carrying system are determined with the three-dimensional solid element by using SAP2000 program [3]. Finally, the displacements and stresses under self-weight and earthquake loading are obtained. The general structural behavior and the dynamic characteristics of the building are demonstrated.

2. THE STRUCTURE OF PANAGIA TON ISODION CHURCH

The principal elements of the basic system of the building are the small dome at Narthex, the piers and walls at the outer front of the building, the interior piers, vaults spanning between all the piers, and three semi-domes at Ieron. The secondary system can not be considered independently from the primary structural system, including circular piers at Klitos, and towers at the corners. The church, which includes the main entrance, Narthex, is in the shape of rectangle in plan. At the outer front of Narthex and inside there exist piers. The piers are connected by vaults in two directions. There is a dome at the middle of Narthex, next to the entrance door. In the north and south fronts the stone walls exist. These walls are connected to the interior square piers by vaults in two directions. There are other piers which have circular cross-section at Soleas. The circular columns at Soleas contain a wooden pier inside in 30x30 cm. square cross-section, which was coated by a lime-mortar, which contains brick particles inside. There are a large scale semi-dome and two small scale domes at Ieron, in the east part of the building. There are two towers at the north and south corners. The north one is bell tower, and the stairs at the end of the Klitos go to this tower, and the stairs at the south open to a place used as store. The structural system of the building is shown in Figure 1.

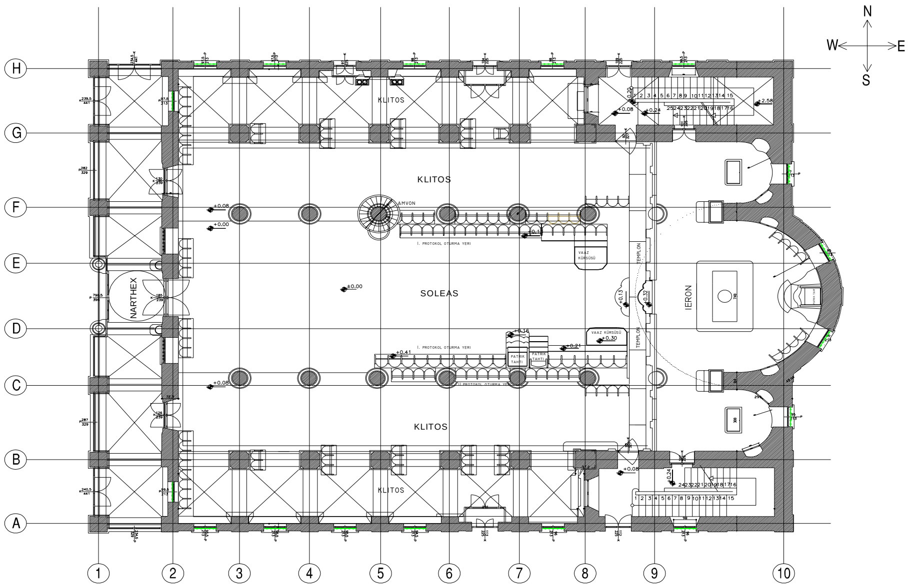


Figure 1 The structural system of Panagia Ton Isodion Church

3. MATERIALS PROPERTIES OF THE CHURCH

For the assessment of material properties of the building, horizontal core samples are taken from outer façade walls and from some piers inside of the church. The core samples are cut and dried in an oven. Dried core samples are tested for physical and mechanical assessment. Unit weights of the masonry materials are shown in Table 1. Static moduli of elasticity and compressive strengths of the cores are determined and test results are shown in Table 2.

Table 1 Physical properties of samples

Sample	Unit Weights (KN/m^3)
Kufeki	20.21
Limestone	26.85
Brick	14.27

Table 2 Mechanical properties

Sample	Compressive Strength (N/mm^2)	Modulus of Elasticity (N/mm^2)
Kufeki	16.4	5100
Limestone	38.5	15800
Brick	7.8	1300

In this research, masonry unit samples are tested instead of masonry. Mechanical material properties, that is necessary to understand its structural behavior under the action of static and earthquake loadings, are calculated from the units of masonry [4]. The working strengths (stress for safety) are calculated by using the approach given in “Specifications for Structures to be Built in Disaster Areas, 2007, Section 5” [5]. The compressive strength and working compressive strength of the wall are calculated as 50% and 25% of the unconstrained wall strength, respectively (Section 5.3.2c) [5]. By using the material strength properties of the building given in Table 2, the wall working compressive strength are calculated and given in Table 3.

Table 3 Wall working compressive strength

Wall	Working Compressive Strength (N/mm^2)
Kufeki walls	2.05
Limestone walls	4.81
Brick walls	1.00

The working tensile strength of the material is taken as 10% of the working compressive strength given in Table 3. The moduli of elasticity of 10000 N/mm^2 for wooden piers [4] and a Poisson’s ratio of 1/6 for all elements can be used. The shear stresses found in the wall for earthquake analysis will be compared with the working shear strength. Working shear strength is calculated by the following formula given in "Turkish Earthquake Code, 2007" [5].

$$\tau_w=\tau_o+\mu\sigma \tag{1}$$

Where τ_o is the wall cracking shear stress, μ is the friction coefficient, σ is the wall vertical stress, respectively. Thus,

Working shear strength for stone walls; $\tau_w = 0.10 + 0.5\sigma$
Working shear strength for brick walls; $\tau_w= 0.15 + 0.5\sigma$ (2)

is obtained [5].

4. STRUCTURAL ANALYSIS

In the analysis, the entire building is modeled by using SAP2000 Program. In the analysis, the solid element, which is in three- dimensional and 8 nodded-brick type, is preferred and the values of stresses are given at the nodal points with respect to a global coordinate system.

The analytical model contains all essential elements of the structural system; i.e., the piers and the wall at the exterior front, the interior square piers and circular piers at +0.00 code, the upper part of those square columns which turn into circular cross-section at the code of + 4.82, the vaults which connect the piers in two directions, the dome exist in the main entrance (at Narthex) in the west part and finally the semi-domes at the east front (at Ieron). The roof of the church is included into the system with making idealization as frame element. The circular piers at Soleas are taken as square cross-section in the modeling and tension bars connecting these piers are defined as frame element. In addition, in the modeling, the window openings in the building are also considered. On the other hand the bell tower at the north front and the tower at the south front are not included in the modeling of the building, they are considered separately.

In order to define the geometry of the structural system, especially those with curved structures like domes, semi-domes and vaults; finite element mesh is established by using AutoCAD Program. For the entrance level (+0.00 code) and suspended floor level (+4.82 code) a model is programmed, and the finite element mesh formed by AutoCAD Program is transferred to the SAP2000 Program. In the finite element modeling, a mesh consisting of 501 frame elements and 30722 solid elements is chosen and 47992 joints are defined by this mesh. Although the peak stresses obtained in FEM are mesh dependent for small number of elements, the number of elements used in the presented model is high enough to accept the results accurate. The details of the three-dimensional modeling are given in Figure 2.

The static and dynamic analyses are made by entering the load systems as follows;

$$G+Q,\ G+Q+Ex,\ G+Q-Ex,\ G+Q+Ey,\ G+Q-Ey \qquad (3)$$

Where G, Q, E show self-weight load, live load and earthquake effect respectively, and in the calculations Q is omitted because it is small compared with G.

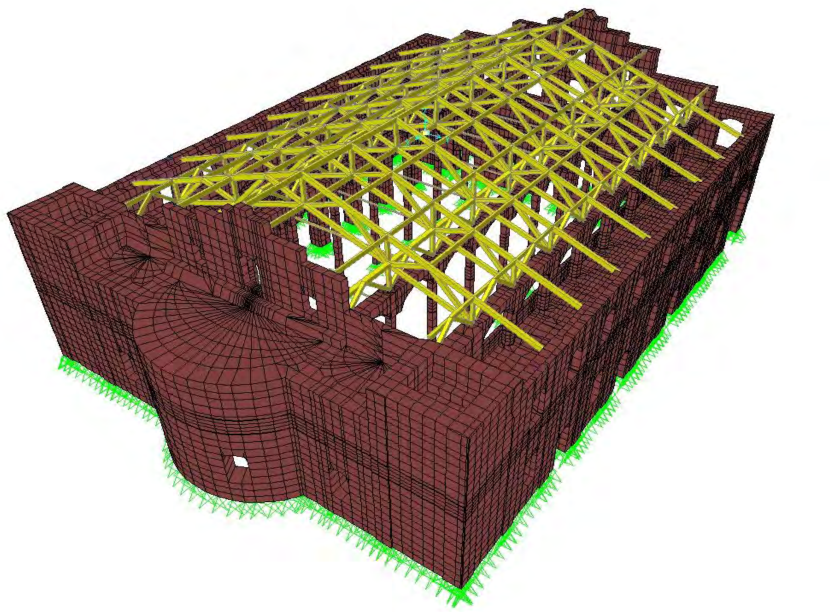


Figure 2 Solid finite element modeling of the church

4.1. The vertical compressive and tensile stresses under self-weight loading

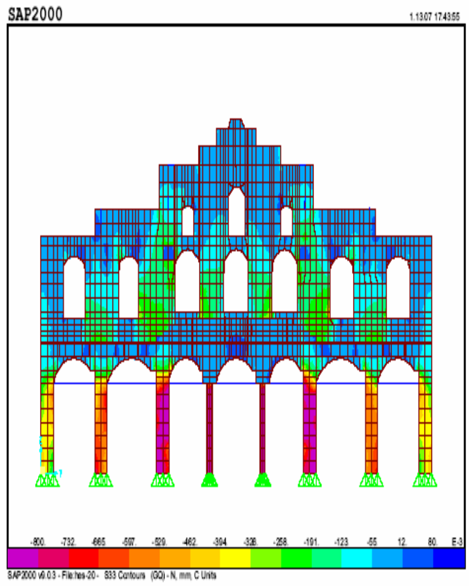
For the load combinations given in eq. (3), the displacements and stresses under self-weight loading are obtained by the analysis. Internal stresses for self-weight loading condition are shown in Figure 3. For self-weight loading, the mentioned controls are made as follows:

Two square piers in the edge Line 1 have compressive stress about 0.85 N/mm^2 and two marble piers have that of about 1 N/mm^2. These compressive and tensile stress values are under the limits; therefore they do not posses any problem (Figure 3a).

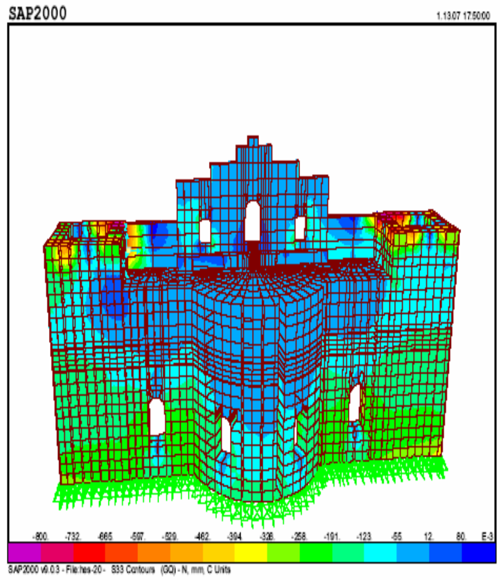
It is observed that the compressive stresses born in the Ieron side remain under the compressive working strength. However, the tensile stress formed in the wall at the upper part of the small domes reaches the level of 0.085 N/mm^2 in a small region (Line 10). This value is also under the required limits (Figure 3b).

In the edge line at the south front of the building (Line A), there is no problem with respect to compressive and tensile stresses. Only, the tensile stresses in a small region of the window edge at the upper floor in the tower side exceed the working tensile strength (0.085 N/mm^2); however this value is under the limits too (Figure 3c).

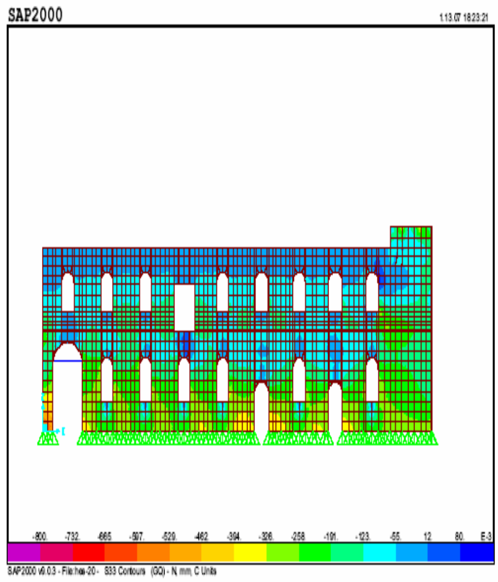
For the square piers in Line B, the compressive stresses in the woman gathering-place level reach up to 0.85~0.9 N/mm^2. This value remains under the compressive stress limit for the brick walls, and also the tensile stresses are under the limits (Figure 3d).



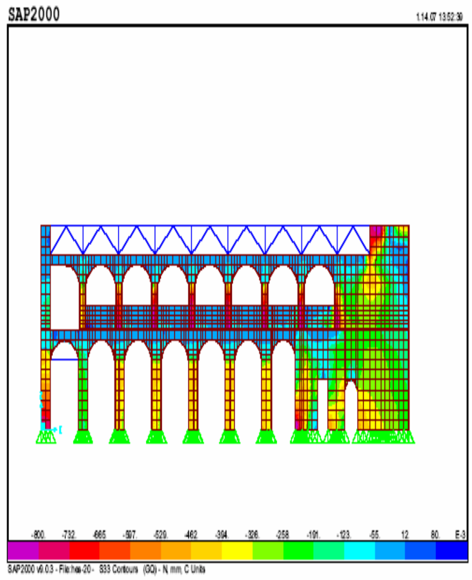
a) Line 1



b) Line 10



c) Line A



d) Line B

Figure 3 Stresses under self-weight loading

4.2. Shear stress controls under earthquake loading

Turkish Earthquake Code requires shear stress control under earthquake loading in the masonry buildings due to the shearing is being the most critical stress situation. The periods of the building in x-direction (the movement of triangular walls rising up to the roof in the front and back façades) and y-direction (the movements of the square piers and the vaults connecting the 30x30 cm. square columns) are found as T_x=0.279 seconds and T_y=0.232 seconds, by dynamic calculation (modal analysis). In determining the earthquake loads, "Turkish Earthquake Code, 2007, Section 5" [5] is used; accordingly the static earthquake load coefficients in x and y directions are taken as 0.375. A_o=0.30, $S(T_1)$=2.50, I=1 and $R_a(T_1)$=2 are used in the calculations. The self-weight load of the building is calculated as 23018 KN. Hence, the equivalent static base shear force is found as 8632 KN by using the concepts in "Turkish Earthquake Code, 2007" [5]. The mode shapes of the building obtained by modal analysis are given in Figure 4.

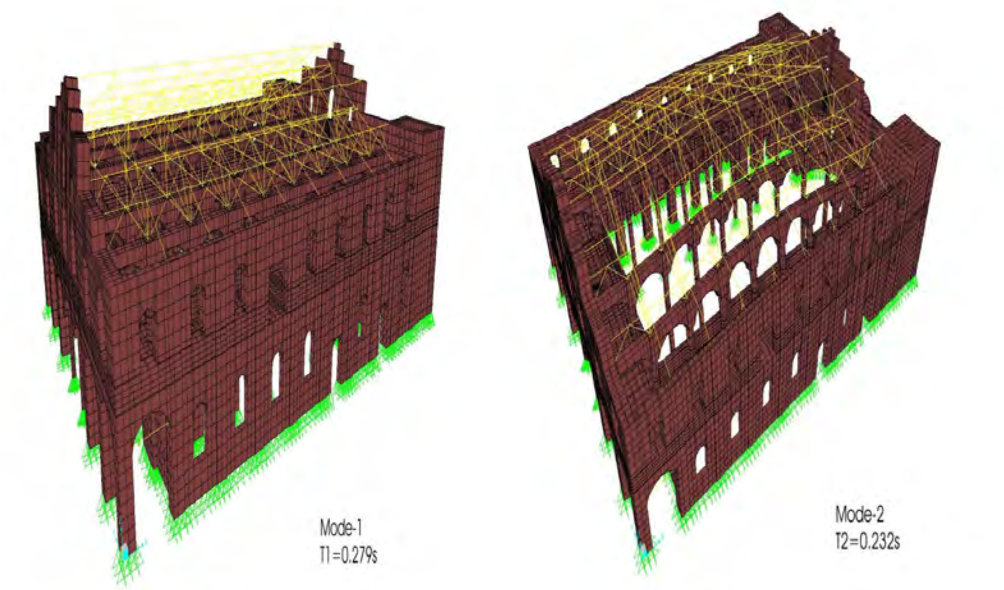


Figure 4 Mode shapes of the church

The shear stresses formed due to the earthquake loading in the parallel directions to the horizontal joints of the walls are calculated at the entrance (at the +1.365 code) and upper floor (at the +5.91 code) levels. The wall shear stress is calculated by considering the floor shear force. To calculate the wall shear stress, the earthquake force on the wall is divided by the cross-sectional area, and the shear stresses are compared with the working shear strengths obtained by using the relation (τ_w =0.10 +0.5σ) in the Table 4 given below. The wall shear stresses are found smaller than working shear strengths as shown in Table 4.

Table 4 The wall shear stresses due to earthquake loading

Code and Line	The Wall Shear Stress	The Working Shear Strength
+5.91 Code, Line A	0.0292 N/mm^2	0.039 N/mm^2
+5.91 Code, Line F	0.0284 N/mm^2	0.038 N/mm^2
+5.91 Code, Line 1	0.0365 N/mm^2	0.049 N/mm^2
+5.91 Code, Line 10	0.0322 N/mm^2	0.043 N/mm^2
+1.365 Code, Line A	0.0940 N/mm^2	0.125 N/mm^2
+1.365 Code, Line F	0.0910 N/mm^2	0.121 N/mm^2
+1.365 Code, Line 1	0.0820 N/mm^2	0.110 N/mm^2
+1.365 Code, Line 10	0.0810 N/mm^2	0.110 N/mm^2

5. CONCLUSIONS

The historic Panagia Ton Isodion Church is analysed by using SAP2000 Program with the solid element to determine the structural behavior for the self-weight and earthquake loading. By examining the structural analysis results, the observations made of the building and considering the method in 2007 Turkish Earthquake Regulations, it is seen that both the compressive and tensile stresses do not exceed the required working stress limits. As can be seen from the calculations made for the codes of +5.91 and +1.365, the shear stresses are well below than the shear working strength. Large deformations under the earthquake loading are seen at the gable walls on the east and west façades of the church. Special cautions should be taken for the gable walls during the restoration. There is no risk in the brick and stone-made part of the building. However, it can be advised that only decayed wooden piers and cast-iron tension bars can be changed during the restoration of the building, with respect to the safety of the structure.

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STRUCTURAL ANALYSIS OF ISTANBUL BEYAZIT II MOSQUE RETROFITTED BY MIMAR SINAN

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ABSTRACT

In this paper, historical sequence of semi-domed structures in Anatolia is given and relationships among Hagia Sophia Museum, Beyazit II Mosque and Suleymaniye Mosque in Istanbul have been emphasized. The current situation of Beyazit II Mosque, which was retrofitted by Mimar Sinan between 1571 and 1574, has been evaluated. Retrofitting of the case study mosque, Beyazit II, is inspected in terms of earthquake engineering aspects. Basic definitions for similar structures, such as reliable modelling resolution, modelling aspects, earthquake loading have been defined. 3D Linear analyses results of Beyazit II Mosque, after and before the retrofitting are demonstrated and evaluated.

1. INTRODUCTION

The survived, repaired, retrofitted, reconstructed, damaged or even totally collapsed Roman, Byzantine and Ottoman structures i.e. the historical monuments in Istanbul are a kind of lecture notes for new generation earthquake engineers. Istanbul Beyazit Mosque, which was completed in 1506 and retrofitted between 1571-1574 by Mimar Sinan, is one of the most important standing evidences in the world for the history of earthquake engineering. The mosque experienced a strong earthquake just 3 years after its opening and was repaired. During some strong earthquakes in the following years, the mosque was retrofitted by adding some external jacketing to the columns, with anchorages and a modified cut-stone arch system. The mosque is also important since it is the first mosque which was constructed with the same structural type (two semi-domes) as Hagia Sophia after the conquest of the city in 1453.

The structure of Hagia Sophia of Istanbul, built between 532 – 537, can be accepted as the most daring and light spatial structure of the early Medieval Age. Just after the conquest in 1453, some trials have been conducted in order to mix Ottoman knowledge and Hagia Sophia. Thus, well known historical mosques of Istanbul such as Fatih and Beyazit II were constructed. Although it was constructed as a technological revolution, Hagia Sophia had some structural weaknesses [2, 5, 6, 7] and these weaknesses were repeated during the construction of first mosques just after the conquest, including Beyazit Mosque. The most important deficiency is the weaker main arches under the semi-domes in Hagia Sophia and also in Beyazit Mosque (Figure: 1).

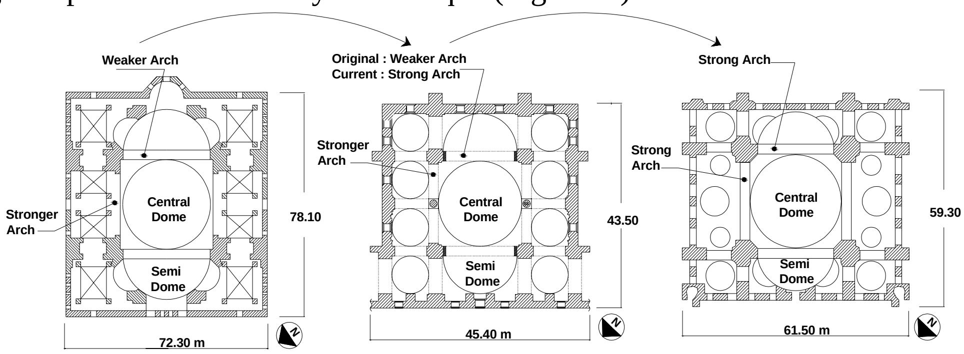

Figure 1

Layout plans and arch positions of a) Hagia Sophia, b) Beyazit Mosque, and c) Suleymaniye Mosque

2. A STRUCTURE RETROFITTED 433 YEARS AGO: BEYAZIT II

The soil profile is layered as 6m infill / 17m green clay / 9m silty clay / 16m green clay / cracked greywacke. Lav [5] has reported that, for an earthquake occurred in a source approximately 20 km far away from the mosque and has a PGA between 0.25 – 0.35g, it is expected maximum 4-5 times amplification for structures which are between 0.33 – 0.50 seconds in the natural period range such as Beyazit Mosque.

Original structural system of Beyazit Mosque is a combination of Hagia Sophia as four main arches and semi-domes, and Ottoman construction culture. The mosque contains four great brick and cut-stone composed arches springing from stone piers that offer primary support for a central dome with 16.78m diameter and 36.5m height and two semi-domes (Figure: 2). Main arches under semi-domes had 90cm depth and 240cm width initially, however, after retrofitting they have approximately 180cm depth, and perpendicular strong arches have almost the same dimensions as current arches under semi-domes. Furthermore, the upper half of the stronger arches is made of brick while lower parts are made of cut-stone. The original section of the weaker arches was 90cm complete brick section but a cut-stone strong arch and stone filling elements were added to that original arch (Figure: 3-4).

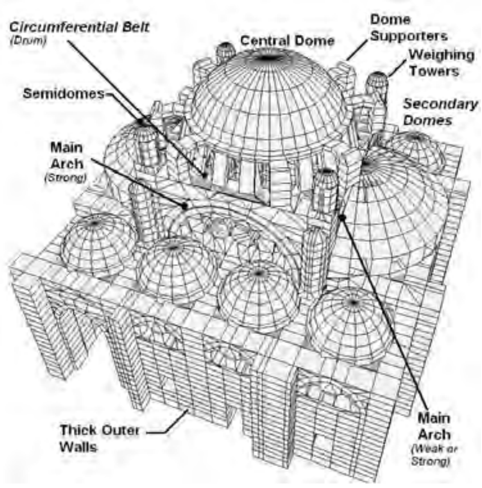

Figure 2 The most common elements of a classical Ottoman mosque

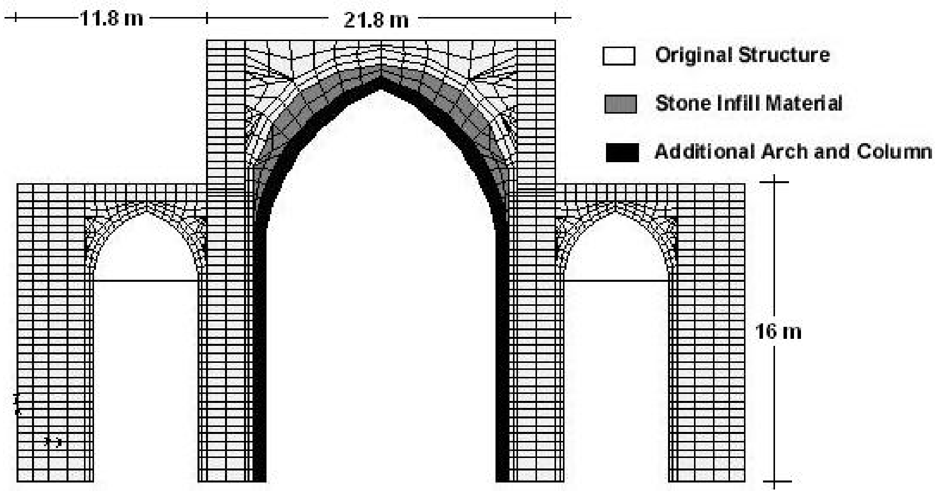

Figure 3 Original and additional elements of the weaker arch

Material properties of the structure have been taken from an ongoing research project [9] and defined in Table 1.

Dynamic properties of the mosque in terms of damping and natural frequencies have also been captured by the aforementioned experimental studies [8]. Natural frequencies are found 2.6 Hz for both x and y lateral directions and 4.0 Hz for the torsional movement. Damping is also determined by experimental tests as 16% for the first mode. The damping value of 16% seems surprisingly high, however, it should be noted that historical structures which are made of massive masonry material may have much different dynamic properties than modern RC structures. Additionally, it is known that the damping values have tendency to decrease with the increasing amplitudes of the shaking. Deterioration of the structure leads to much smaller damping values [3]. For that reason, earthquake analyses of the structure should be conducted by a lower value of damping (i.e. 4 %), which is average of the damping values measured at Hagia Sophia and Suleymaniye during moderate strong motions[3]. This issue needs to be examined in more detail.

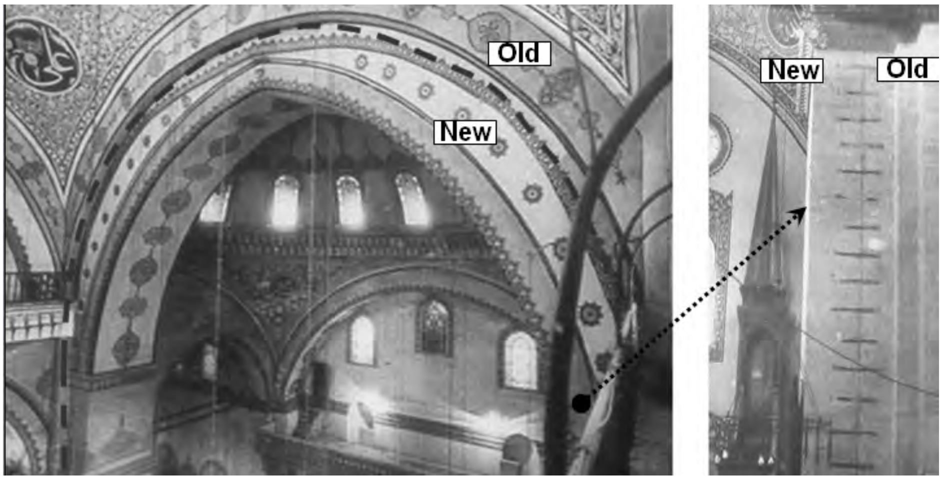

Figure 4

Original and additional arches and anchorages

Table 1

Material properties used in modelling

Material properties	**Modulus of elasticity** *(MPa)*	**Poisson ratio**	**Weight per unit volume** *(t/m^3)*
Brick arches	3 500	0.20	2.20
Piers and stone arches	14 830	0.18	2.40
Brick domes	3 500	0.20	2.20
Brick pendentives	3 500	0.20	2.20
Brick semi-domes	3 500	0.20	2.20
Granite columns	17 000	0.15	2.60
Iron ties	180 000	0.30	7.85

The sources report that during 1509 earthquake the main dome of the mosque was shattered, other domes and arches of the complex split, and its store-room and minaret collapsed and the newly built structure was repaired rapidly. Although earthquake catalogues give emphasis on some other earthquakes between 1509 and 1571, mosque's retrofitting starting date, no other data could be found about any damage till 1719 earthquake, which had surface wave magnitude Ms=7.6 [1].

3. EFFECT OF THE RETROFITTING AND ANALYTICAL MODELS

As mentioned by himself in *Book of Architecture* (*Tuhfet'ul Mimarin)*, Mimar Sinan added a strong arch to the mosque and as investigated in this paper, the effect of that simple arch is significant [10].

3.1. Earthquake engineering aspects of the retrofitting

As shown in Figure 3, a steeper arch was added rather than a perfectly circular arch. An individual analysis has been conducted during that study in order to understand this different application and it has been proven that the steeper arch, which touches the main piers at the level of 3.5m lower than the original arches, generates almost 21% less horizontal force and 31% less moment effect on main columns due to vertical loads.

Anchorages between old and new columns are also interesting since they have similar mentality to modern column jacketing and anchoring techniques. Anchorages have 45cm length, 4cm width and approximately 2cm thickness and are installed in every 45cm. Physical fixing via melted lead was used as modern chemical fixing with epoxy material is used in our era.

Although it seems that there are 3 different layers of arches in the current situation, exposed places by loss of the cover show that there is 10 – 15cm thin brick layer between brick and stone arches. That thin layer provides a perfect connection between the rough surface of the old brick arch and perfectly smooth surface of the stone arch.

3.2. Linear FEM analysis

3D computer models of the original and retrofitted current structure have been created. The foundations have been modelled as fixed and at the level of -2.5m. The created acceleration spectrum for 4.0% damping gives a maximum spectrum coefficient of 2.75, while the Turkish Earthquake Code of 1998 gives 2.50 for the plateau of the spectrum which is built for 5% damping. The Earthquake Zone Acceleration Coefficient (A_0=0.40g) from Turkish Earthquake Code of 1998 and a lateral load reduction factor (i.e. the behaviour factor, q=2.5), based on engineering judgment, have been used for earthquake analysis in order to create a benchmark decision. An elastic modal superposition analysis has been conducted on full 3D model by employing the earthquake spectrum defined in the Turkish Earthquake Code of 1998. The comparison between the situations before and after the retrofitting is given in Table 2.

One of the most important problems during 3D analysis is the sizing of the elements in the constitutive model. Before performing analysis, an individual size effect study has been conducted by modelling only a single frame of the mosque. The linear constitutive model has been created with elements which have $1m^3$ average volumes. There is a dramatic drop of accuracy after $1m^3$ element volume, because of the sudden change of the arch geometry *(the bigger elements require* *non-circular arches).* For the analysis of such massive monumental buildings, authors suggest to conduct a preliminary analysis, at least on a part of the structure, to obtain a reasonable meshing size.

The elastic FEM analysis, in spite of not describing the actual behaviour, provides valuable information about the tensile zones, the possible crack formation and for the case of Beyazit Mosque, an approximate idea about the structural effects of the retrofitting. The variation of periods, displacements of arches, columns and the main dome have been investigated in that study.

There is a general decrease of periods after retrofitting, except the first period in the perpendicular direction to the retrofitted arch (Figure: 5). This phenomenon appears because of the difference between the increase of total mass and the increase of rigidity in the direction of the first mode. Since the first mode direction is the out-of-plane direction of the retrofitted arch, it has mass contribution more than rigidity contribution. Another observation about the periods is that the third period, which refers to the lateral squeezing, decreased 15%, since that movement is strongly related to arch rigidities. It is also found that the analytical periods are slightly lower than the experimental ones. The reason behind that might be the existence of cracks and the soil-structure interaction which have not been included in the model.

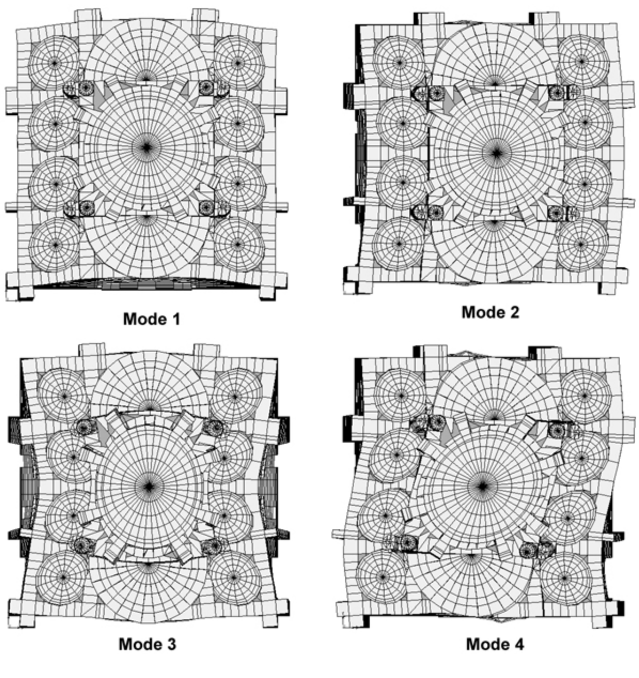

Figure 5

First four mode shapes (pre- and post-retrofitting cases have similar shapes)

The maximum displacement decrease is observed in the displacements of weaker arch in the retrofitted direction. That highest displacement decrease is strongly related to both the increase in the cross-sectional area of the column and the new geometrical form of the steep arch as mentioned before. After the retrofitting, the lateral displacement of the central dome and the top of the columns are almost a quarter less than the one of the original case.

Table 2

Comparison of the linear FEM analysis results for before & after the retrofitting

Periods *(sec)* **and displacements** *(m)*	**Before**	**After**	**Difference** *(%)*
First four periods *(sec)*	0.30 / 0.30/ 0.22 / 0.19	0.31 / 0.29 / 0.19 / 0.19	3 / -3 / -15 / 0
Weaker (retrofitted) arch displacement in 'x' (1)	0.0420	0.0276	-34
Weaker (retrofitted) arch vertical displacement	0.0038	0.0030	-21
Column displacement in 'x'	0.033	0.0247	-25
Central dome displacement in 'x'	0.0508	0.0366	-28
Central dome vertical displacement	0.0042	0.0035	-17
Differential settlement of dome base	0.0058	0.0006	-84

(1) X indicates the direction parallel to the retrofitted arch.

4. CONCLUSIONS

In reference to Beyazit mosque, retrofitted by Mimar Sinan, in this paper a short comparison among Hagia Sophia Museum, Beyazit and Suleymaniye Mosques is presented. Weaknesses of Beyazit Mosque have been investigated and the results of the linear analysis have been evaluated.

It must be noted that the authors suggest creating an initial model before continuing with analyses of main model, which is a part of the monument and conducting preliminary analyses with different element resolutions in order to decide the accuracy of the constitutive model.

The periods of the linear 3D FEM analyses are slightly lower than the experimental ones because of existing minor and major cracks on secondary domes, semi domes and even on the central dome and the main arch. It must also be emphasized that any model with elastic springs representing the soil would give higher periods as in the experimental test results.

In conclusion, analyses result that the weak arch deficiency of the presented Beyazit Mosque was corrected by retrofitting the structure with an additional arch, infilling arch and an extension of cut stone columns. It is found in linear analysis that retrofitting caused 21% decrease in vertical and 34% decrease in lateral displacements of the weaker arches in x (parallel to weaker arch axis) direction earthquake loading. Central dome displacements are also decreased, i.e. 28% in lateral and 17% in vertical directions. The most impressive improvement by the retrofitting is observed in the differential settlements of the dome base. The differential settlement is decreased 84% by the retrofitting.

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TOWARDS A METHODOLOGY FOR SEISMIC ASSESSMENT OF MONUMENTS: THE CASE STUDY OF SANTA MARIA OF BELÉM CHURCH

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ABSTRACT

This paper presents an integrated methodology for the seismic behaviour characterization of ancient masonry structures of significant cultural heritage importance, towards the mitigation of the seismic risk. The proposed methodology is here applied to an outstanding case study composed by the Santa Maria of Belém Church, in Lisbon, Portugal. This Church was built during the 16th century and is included in the Monastery of Jerónimos compound, which is one of the most emblematic Portuguese monuments.

The methodology includes the following main steps: local seismic action characterization including site effects; simple numerical modelling for a preliminary knowledge of the structural behaviour; experimental mechanical characterization of materials and structural elements; establishment and installation of static and dynamic monitoring systems aiming at a better understanding of the static and dynamic behaviour; development of advanced numerical models including the necessary calibration against relevant experimental data; non-linear dynamic analysis of the structure for different earthquake levels. According to the non-linear dynamic analyses performed in the time domain, the results obtained so far show that the monument will be under great stress against a far-field earthquake as strong as M = 7.4 (475 yrp), able to cause damage but neither local or global collapse are to be expected.

1. INTRODUCTION

Ancient masonry structures are particularly vulnerable to dynamic actions, especially to seismic ones. Due to the ageing process, as well as to environmental factors, many cultural heritage buildings, as structures planned and constructed in the past, are vulnerable to earthquakes, which may unpredictably induce a collapse of a portion of the building or drive the whole structure to a rapid failure. However, anti-seismic design requires the definition of the local seismic action (a rather challenging issue) and knowledge about the characteristics of existing buildings.

The peculiarities of ancient masonry constructions require an approach similar to those used in medicine, including stages as [4]: (a) anamnese, (b) diagnosis; (c) recommendations for conservation or design reinforcement; (d) intervention; (e) evaluation of the effects of the intervention (e.g. monitoring of the structure). Based on these ideas, the proposed methodology prescribes a set of experimental and numerical procedures for the seismic behaviour characterization as a support for conservation recommendations or for possible strengthening measures towards the mitigation of the seismic risk. The methodology should be understood as a dynamic and interactive process. Due to the modern context of minimum repair and observational methods, NDT techniques including monitoring procedures are particularly attractive.

Considering that each historical building is unique, it is really important that each structure should be studied accordingly. In this sense, the proposed methodology is here applied to an outstanding case study composed by the Santa Maria of Belém Church, a part of the Monastery of Jerónimos compound, which is one of the most emblematic Portuguese monuments.

The paper is organized as follows. In section 2 the approach proposed for the seismic assessment of monuments is described. Afterwards, in section 3 the methodology is applied to the case study. Finally, in section 4 the first results and conclusions are discussed.

2. SEISMIC ASSESSMENT OF MONUMENTS

The seismic safety assessment of historical constructions is a quite complex task. In particular, little is known about materials and variability of its mechanical properties, existing damage, constitution of the inner core of walls, columns and vaults, among other relevant difficulties. Usually, this task cannot be carried out without combining historical data with an experimental and analytical investigation.

A proper methodology should be initiated with a preliminary investigation, interpretation of historical documentation, understanding of the historical context and appraisal of the general structural characteristics of the construction (anamnese). Then, a second step might deal with a preliminary diagnosis using simplified methods whose application is based on geometric data manipulation of structural masonry walls and columns to produce scalar indexes for in-plane or out-of-plan safety. A detailed description can be found in [8] and [11].

Afterwards, a more rigorous assessment of the current safety conditions is necessary. Nowadays, an advanced safety assessment of existing structures is, in general, based on numerical analyses. However, no reliable modelling is possible without proper insight into the relevant characteristics of the structure. For that purpose and given the complexity of these constructions, an integrated program of tests is proposed, involving the following tasks: (a) detailed visual inspection (aiming at locating critical zones with damage or other irregularities); (b) soil foundations survey (aiming at a geological and geotechnical characterization); (c) definition of a set of experimental, in-situ and laboratory works.

All these tasks are essential for the definition of further actions and for the implementation of monitoring programs. In what concerns to site inspection and evaluation techniques, today's technology makes available a vast range of equipments and techniques, mostly non-destructive or low invasive, which enable the gathering of data required to feed and validate numerical models, e.g. see [1] and [6]. Additional laboratory tests can also be carry out on material samples for mechanical, chemical and physical characterization.

Within the framework of this methodology, both dynamic identification and monitoring are fundamental NDT techniques to be used. In the case of historical constructions, these aspects are highlighted due to the importance of the structure. The dynamic identification provides the modal parameters of the structure and long term monitoring systems, with continuum data record and seasonal analysis, are important complementary tools for any experimental or numerical research as they allow satisfying the following purposes: (a) better understanding of the complex structural behaviour of the construction; (b) identification of any possible progressive phenomenon; (c) detection of damage at an earlier stage; (d) calibration of boundary constrains and global stiffness of numerical models under development; (e) assessment of environmental influences in the structural behaviour; (f) effectiveness assessment of possible strengthening/repair works.

Another important and challenging issue for advanced safety assessment of monuments is the local seismic action characterization. This characterization should be done in accordance with regional seismic hazard and local geotechnical conditions. Besides the attenuation conditions of the seismic signal from distant source sites, the local ground motion amplifications are of great importance, e.g. see [5] for further details.

The next step is deeply related with the numerical modelling and structural analysis of the construction. In many ancient constructions, the borderline between architectural details and structural elements is not always clear. The geometrical complexity of structures increases the difficulty in defining a finite element model appropriate for structural analysis. Therefore, any numerical model to be adopted should not be excessively complex, especially when non-linear dynamic analyses in time domain are to be used.

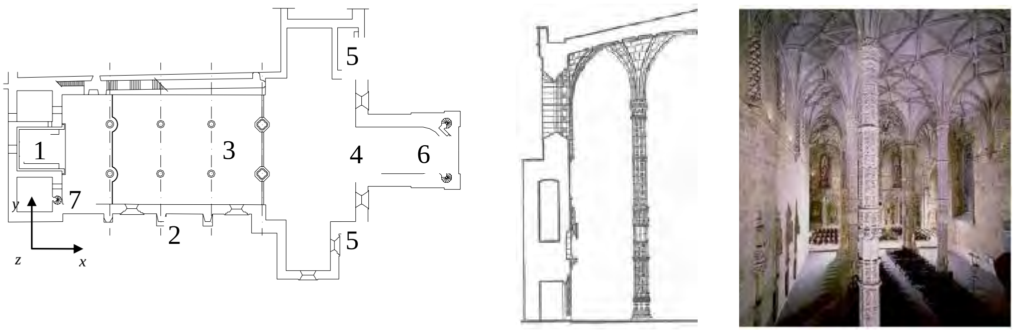
The validation/calibration process should ensure that a reliable simulation of the present condition of the structure is achieved. Therefore, this process involves an interactive development in which the behaviour of the structural elements is checked step by step against experimental results. However, a preliminary elastic static analysis prior to any non-linear dynamic analysis is of great interest to identify the main structural vulnerabilities and to define strategies for subsequent analysis. In general, to avoid conclusions of doubtful reliability, a structural observation over a large period of time, with proper monitoring systems, seems to be necessary to validate preliminary analyses of results (qualitative, quantitative and experimental).

The structural intervention, if really necessary, should be done observing the cultural and historic significance of the construction, without inducing significant changes to the structure (minimal intervention) and with adequate materials and techniques. The post-intervention assessment of the effects produced is of crucial importance. At times, the difficulty of evaluating the real safety level and the possible benefits of interventions may suggest "an observational method", i.e., with iterative and step-by-step approaches, starting from a minimum level of intervention, with the possible subsequent adoption of a series of supplementary or corrective measures, see [4] for further details. After the intervention, a monitoring plan is also required, most of the times using the same monitoring systems installed previously.

3. APPLICATION TO A CASE STUDY

The Monastery of Jerónimos, dating from the 16th century, is, most probably, the crown asset of the Portuguese architectural heritage. One of the courts is composed by the Church and the cloister of the monastery. The Church has considerable dimensions, namely 70 m long and 23 m width, see Figure.

The main nave is divided by two rows of slender columns, with a free height of about 16.0 m. The transverse sections of the octagonal columns in the nave have a radius of 1.04 m with top fan capitals that reduce the effective free spans of the slightly curved vault, see Figure. For additional information see reference [3].



(a) (b) (c)

Figure1. Santa Maria of Belém Church: (a) plan (1-axial doorway, 2-lateral doorway, 3-nave, 4-transept, 5-side chapels, 6-chancel; 7-South bell-tower); (b) half of transversal cross-section; (c) view of the three naves.

The construction resisted quite well to the 1755 Lisbon earthquake. In the following year (December, 1756), a new earthquake caused the collapse of one column of the nave and a partial ruin of the high choir [9]. Later on, in 1887-1888 the bell-tower was modified and elevated. In 1947-1949 the Church covering was restored and brick masonry walls were built at the extrados of the vault nave to provide support for tiles. In 1963, minor consolidation works were performed including the vault bed joints refill. Since 1949, several historical documents have referred stone fragment falls from the vaults of the Church. These successive happenings illustrate clearly the need for a reliable seismic assessment of the monument. The analysis of previous existing works allows concluding that the geometrical survey of the main nave demonstrates a vertical non-alignment for all the columns and the external walls. Also, the radar investigation and ultrasonic tests carried out show that the columns of the nave seem to be made of a single block or two blocks [3] and a variable thickness mortar layer seems to exist on the extrados of the vault. On the other hand, a concrete-like material with stones and clay mortar fills the fan capitals [10]. Finally, an existing geotechnical report shows that the bed rock is located a few meters below the surface and that direct foundations were found in the monastery.

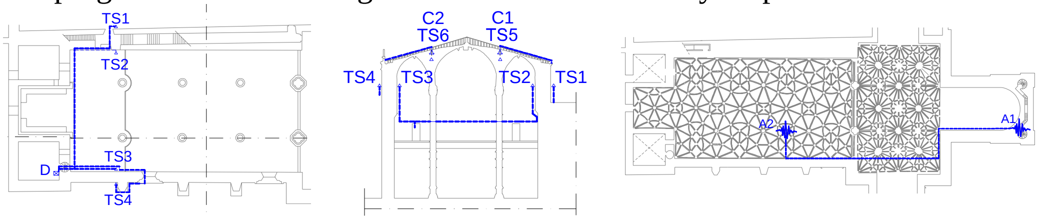
Using available geometric data, a set of simplified in-plane and out-of-plane indexes were computed. The results, summarized in Table , stress the high slenderness of the columns (γ_4) and the apparent vulnerability of the Church in the transversal direction ($\gamma_{3,y}$). For detailed information about these indexes the reader is referred to [8].

Table 1. Simplified indexes based on geometric data.

In-plan area ratio (γ_1)		Area to weight ratio (γ_2)		Base shear ratio (γ_3)		Slenderness ratio of columns (γ_4)	Thickness to height ratio of columns (γ_5)	Thickness to height ratio of perimeter walls (γ_6)
X	Y	X	Y	X	Y			
0.17	0.12	1.2	0.8	1.4	0.95	70	0.06	0.08

A campaign of experimental tests was performed aiming at: (a) mechanical characterization of the materials; (b) dynamic modal identification; (c) long term monitoring through the installation of both static and dynamic monitoring systems. As it was neither allowed to collect samples nor to use flat-jacks, the mechanical characterization of the masonry was performed in laboratory by carrying out uniaxial compressive tests on prismatic limestone prisms, as similar as possible to those observed in the monument. An average compressive strength of 10 MPa and a Young's modulus within the range of 20 - 50 GPa were found.

Both static and dynamic monitoring systems were installed in the main nave of the Church. The static monitoring system (see Figure a) is composed by six temperature sensors (TS1 to TS6), two uniaxial tiltmeters (C1 and C2) and one data logger (D) for the data acquisition and data record. The dynamic monitoring system is composed by two triaxial accelerometers connected to two strong motion recorders. Both recorders are interconnected, which allows a common trigger and time programmed records. Figure b shows the sensors layout position.



(a1) (a2) (b)

Figure 2. Monitoring systems: (a) static monitoring system, (a1) nave plan and (a2) nave cross-section; (b) dynamic monitoring system.

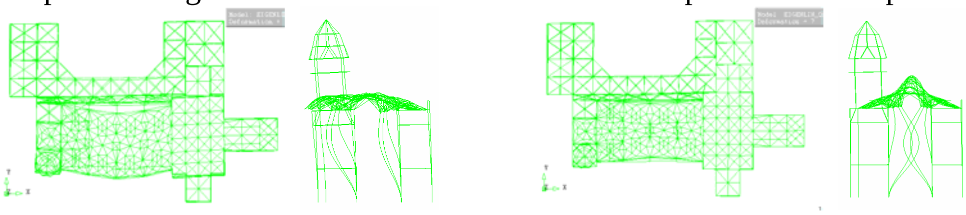
The main nave (vault and columns) of the Church was dynamically identified by resorting to two experimental techniques (EFDD and SSI). Thirty points on the extrados of the vault were selected to measure the acceleration response. Table summarizes the four estimated resonant frequencies, damping coefficients and Modal Assurance Criteria (MAC) for both techniques. The modal identification of the nave columns allow to identify typical first mode shape configurations with a 7.0 Hz resonant frequency.

Table 2. Measured resonant frequencies and damping coefficients for the vault.

Mode	Frequency [Hz]		Damping [%]		MAC
	EFDD	SSI	EFDD	SSI	
1	3.69	3.68	2.34	1.26	0.99
2	5.12	5.04	1.11	2.68	0.92
3	6.29	6.30	1.00	0.82	0.67
4	7.23	7.29	0.77	1.44	0.67

Supported by specific stochastic seismic hazard studies conducted for mainland Portugal, see reference [12], three hazard scenarios with return periods of 475 years (M=7.4), 975 years (M=7.8) and 5000 years (M=8.2) were used. In the absence of available seismic earthquake records, three acceleration time-histories were artificially generated for each seismic scenario using specific generation numerical models, as described in reference [2], resulting PGA values within the range of 0.09g-0.12g, 0.14g-0.17g and 0.21g-0.23g, respectively, for return periods of 475, 975 and 5000 years. Based on an existing geological-geotechnical report, it seems reliable to assume that Jerónimos Monastery is founded on the bed rock. Therefore, no site effects were considered within this study.

For the numerical analysis, a global model of the Church and adjacent structures was developed. Despite the high complexity of the structure, a simplified 3D model composed of beam elements was adopted. A total strain crack model with an ideal plastic material behaviour was adopted with $f_c = 10$ N/mm^2 (compressive strength) and $f_t = 0.01$ N/mm^2 (tensile strength). All the analyses were performed using DIANA 9.1 software [12]. The calibration of the global model was performed in two phases. First, a preliminary comparison against an existing detailed numerical analysis of the vault under its self-weight was performed [7]. A second calibration was based on experimental existing results obtained from the dynamic identification and laboratory tests, presented above. In this way, the Young modulus was assumed to be equal to 30 GPa for the columns and 12 GPa for the other structural elements. The foundation boundaries were kept fixed. Figure illustrates the first and fourth computed mode shapes.



(a) (b)

Figure3. Numerical mode shapes: (a) 1^{st} mode at 3.8 Hz; (b) 4^{th} mode at 5.34 Hz.

A preliminary linear static analysis under vertical and horizontal loads confirmed that the transversal direction (y) of the nave, see also Figure 1(a), controls the behaviour of the structure. Therefore, it was decided to perform non-linear static analyses for both vertical and transversal directions under an increasing gravity load factor until the development of collapse mechanisms. The results from this analysis show the need for a carefully numerical analysis against earthquakes.
Following the methodology presented above, non-linear dynamic analyses were performed for the transversal direction using the HHT time integration method with a time step of 0.01 s. A damping coefficient of 2% was adopted for the computation of the Rayleigh matrix. Up to the moment, only results from the first hazard scenario (475 yrp) are available. The numerical results obtained so far show that: (a) maxima drift is below 0.3%; (b) the average shear base ratio in the y direction is equal to 0.10; (c) important compressive stresses in the vault are observed; (d) the South belfry tower collapse is nearly to happen by overturning; (e) the remaining global stiffness of the structure is about 60% of the original one. According to these results, the Church will be under an important stress state against earthquakes as strong as M = 7.4 (475 yrp) that will cause cracking but neither local nor global collapse is expected. The remaining two and more severe seismic hazard scenarios are currently being analysed and, therefore, no results are available so far. These analyses are of major importance in order to assess the seismic safety of the monument.

4. CLOSING REMARKS

The methodology presented in this paper aims at the seismic risk mitigation of historical structures and it can be used towards the development of management policies for the cultural heritage. The main results, both experimental and numerical, achieved by the application of such methodology to an emblematic Portuguese case study have been presented and discussed.

For the numerical analysis, a 3D model considering both non-linear material and geometric behaviour was developed and calibrated against experimental results. The numerical results concerning the step-by-step seismic analysis for 475 yrp scenarios show that the monument is submitted to a significant stress state that causes cracking, but neither local nor global collapse is reached. However, the collapse of the South bell-tower by overturning is nearly to happen. Two more severe seismic scenarios are currently under analysis.

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INVESTIGATION OF THE SEISMIC RESPONSE OF A BYZANTINE CHURCH – COMPARISON WITH THE EXISTING DAMAGE

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ABSTRACT

In this paper, the investigation of the seismic response of a Byzantine church is presented. Elastic time-history analysis using 3-D finite elements was performed for an earthquake excitation. All the main architectural details of the building were included in the numerical model. In spite of the inherent limitations of the elastic analysis, the results showed good agreement with the existing crack pattern, caused to the building by previous earthquakes. The collapse of the cupola of the main church's dome and of the walls and the dome of the narthex during the 1881 earthquake could also be justified.

1. INTRODUCTION

The analytical prediction of the seismic response of historic masonry structures is a difficult task due to numerous reasons including the complexity of the geometry, the difficulty in modeling the existing damage, the unknown mechanical properties of the materials and the nonlinear behaviour. In order to capture the response of such structures under strong earthquake excitation, nonlinear, time-history analyses are needed. Such analyses require a great amount of computational effort and time while their results are not necessarily reliable because of uncertainties concerning the real nonlinear behaviour of the materials. For this reason, simpler analyses, such as linear static or linear dynamic analysis and static nonlinear (pushover) analysis, are used in many cases, which for certain applications might be adequate if combined with proper engineering judgment. Discussion on these matters has been presented by Lourenço [3], Penelis [4] and Syrmakezis [6].

The application of linear time-history analysis for the investigation of the seismic response of a Byzantine church is presented in this paper. The purpose of the analysis was to understand the dynamic response of the building and to evaluate the effectiveness of the proposed strengthening measures. Due to the complicated geometry, elastic modal analysis and static nonlinear (pushover) analysis were excluded, because it was doubtful whether these methods could capture the details of the response: in the first case, the uncertainties arise from the combination of the modal responses; in the latter, in spite of the fact that nonlinear behaviour of the materials is considered, the accuracy which is usually obtained is low (even for much simpler systems). This happens because dynamic effects are neglected and the calculation of the response is based on the corresponding one for the equivalent single-degree-of-freedom system. On the contrary, linear time-history analysis can capture the main characteristics of the response, but the predicted stresses are not right, since high tensile stresses can be developed for strong excitations that might be larger than the tensile strength of the masonry. In spite of this drawback, however, this method can predict satisfactorily the shape of the deformation and the regions where cracks are expected to form and for this reason it was adopted here.

Since emphasis was given to the behaviour of the weak regions of the structure, a detailed model of the building and the surrounding soil was constructed using 3-dimensional tetrahedral elements, in order to be able to capture the effect of the geometry and determine accurately the critical areas. This model was subjected to a strong earthquake motion, and the regions, where the tensile stresses exceeded the tensile strength of the masonry, were recognized and compared to the existing cracks, caused by past earthquakes. In this way it was verified that the method of analysis was capable of predicting satisfactorily the seismic response. In a later stage, which is not presented here, the proposed strengthening interventions were introduced to the numerical model and the analysis was repeated; thus, the effectiveness of each measure was evaluated.

2. DESCRIPTION OF THE STRUCTURE

All the analyses were performed for the church of “Panaghia Krina” in Chios Island, Greece. The building is composed of three parts: the eastern main church, the middle part, called the narthex and the western part where the main entrance is placed, called the exonarthex. The three parts are statically independent, except for the two E columns of the longitudinal arches of the narthex that are connected at their bottom to the W wall of the main church. The overall dimensions of the building are 21.10 m × 8.10 m. The N view of the church is shown in Figure 1.

The roof of the church consists of vaults and domes. A large dome, compared to the size of the building, covers the central part of the main church. A smaller dome exists over the narthex. The bell tower is located at the western wall of the exonarthex.

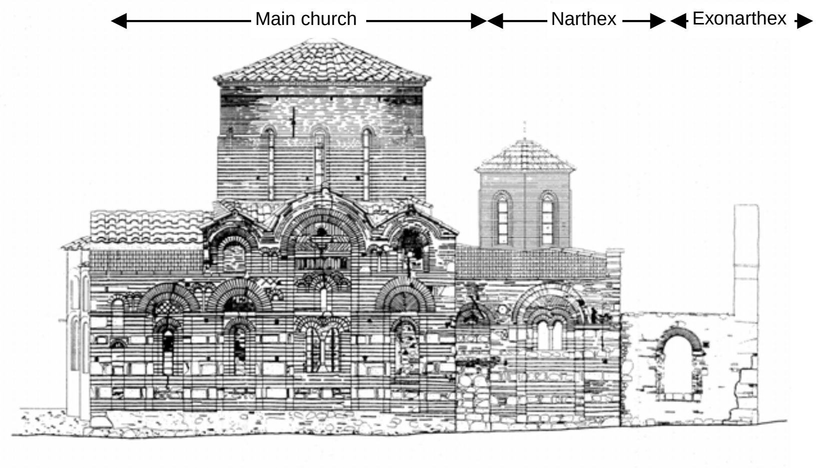


Figure 1: N view of the church (drawing by S. Vogiatzis and M. Paspati, Hellenic Ministry of Culture).

The construction of the main church started in the middle of 11th century and shortly after completion of the works the construction of the narthex started. The exonarthex was constructed in 1747. A detailed description of the church's geometry, manner of construction, particularities and inherent weaknesses, historical and current pathology and intervention designs is presented in another paper submitted to this conference by the Hellenic Ministry of Culture (Miltiadou et al).

There is evidence (e.g. the different time that the murals were painted) that the building suffered severe damage during the 1389 earthquake. Extensive damage was also caused to the structure during the strong earthquake of 1881: the cupola of the main dome collapsed, the roof, the dome and the upper part of the S and N walls of the narthex also collapsed and part of the bell tower fell. After the earthquake, the cupola of the main dome was restored and the narthex was rebuilt. However, extensive cracks still exist at the walls and the roof domes. More severe are those at the domes and the walls of the sanctum, the ones at the W wall of the main church and the cracks at the supporting ring and triangles of the main cupola.

3. NUMERICAL MODEL

The scope of the work presented here was to examine whether linear analysis could capture the main features of the seismic response and predict the existing crack pattern accurately. For this reason, all the details of the structure were introduced into the numerical model, in order to be able to locate the weak zones. The three parts of the building were modeled as statically independent according to the real structure; however, pounding between adjacent parts was not considered. The existing steel ties and timber reinforcements between opposite walls were also included in the model. The walls were assumed elastic with properties: Young’s modulus varying from E=1500 MPa to E=2500 MPa depending on the material at each place, Poisson’s ratio v=0.20 and density ρ=1.90 Mgr/m^3.

The surrounding soil, in an area of 50.00 m × 30.00 m and in a depth of 1.50 m under the foundation, was also considered in order to capture the soil-structure interaction effects. The soil was modeled as elastic material with Young’s modulus E=25 MPa and Poisson’s ratio v=0.25 according to the results of the geotechnical investigation [2]. The depth of the soil in the numerical model was limited to 1.50 m, because this is the mean depth of the soft surface layer above the bedrock. The soil was considered without mass, in order not to vibrate during the seismic motion and, thus, to prevent the development of stationary waves. The seismic motion was applied at the external boundaries of the soil.

The numerical analysis was performed with the code ABAQUS [1] using 3-dimensional, tetrahedral finite elements. For the structure, the maximum dimension of the finite elements was 0.30 m and for the soil 1.50 m. All the walls were modeled with at least two elements along their width. In total, the model consisted of 44,282 nodes and 190,597 elements with 132,348 degrees of freedom. A general view of the model is shown in Figure 2, while in Figure 3 the longitudinal section of the model of the building is depicted.

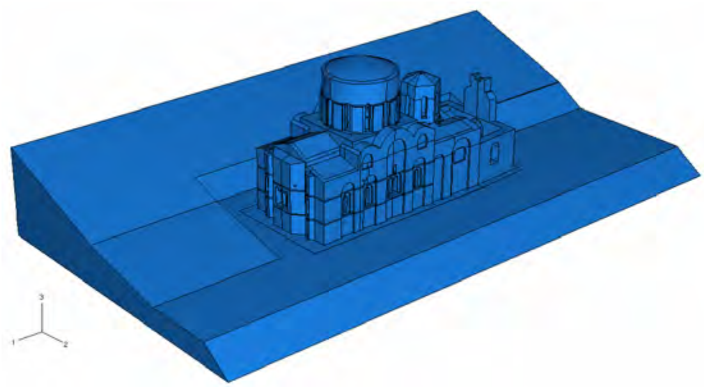


Figure 2: General view of the numerical model.

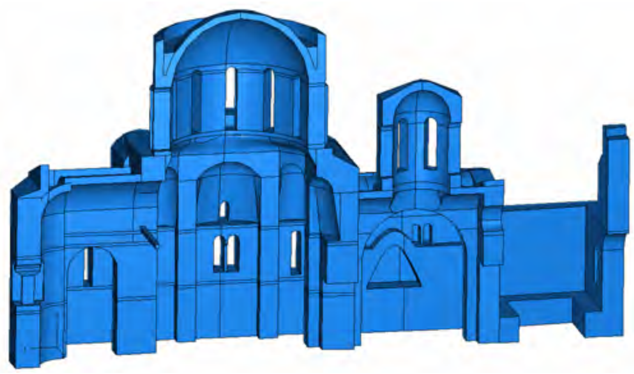


Figure 3: Longitudinal section of the numerical model of the building.

4. NUMERICAL ANALYSIS

For the verification of the assumptions used in the numerical model, modal analysis was performed first and the resulting dynamic characteristics were compared to the results of ambient vibration measurements of the real structure. The comparison was quite satisfactory, with the eigenperiods of the model being slightly smaller than the ones of the building as expected, since the model corresponds to the uncracked structure.
Linear analyses were performed for the gravity loads and for earthquake excitation. In the latter case, modal time history analysis was performed using the 20 first modes and 5% modal damping. Modal time-history analysis was preferred instead of direct integration of the equations of motion, because it requires much smaller run times without losing in accuracy.

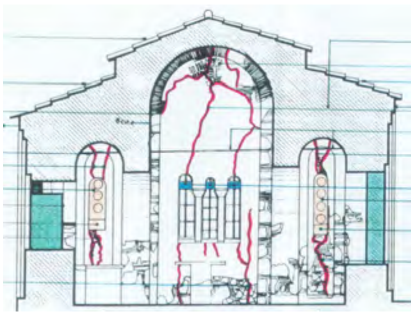
For the seismic response of the system, the Roscoe record of the Northridge, 1994 earthquake, was used as the base excitation. All three components of the record (two horizontal and one vertical) were applied. The stronger horizontal component (pga=430 cm/s^2), which was recorded in a direction parallel to the fault, was applied in the longitudinal direction of the structure (EW), because the known faults in the area are oriented in this direction. The other horizontal component (applied in the NS direction of the church) had pga=262 cm/s^2. Plots of the acceleration time-histories of the horizontal components are given in Figure 4. The corresponding response spectra for 5% damping are shown in Figure 5; in these plots, the elastic design spectrum of the Greek seismic code for ground acceleration *A*=0.30 g and soil type B are also depicted for comparison. The value of *A*=0.30 g corresponds to an earthquake with a return period of about 500 years according to the seismic hazard evaluation that was performed for the area (Psycharis *et* al [5]). It also corresponds to the ground acceleration according to the Greek code with an increase of 25% due to the proximity to seismic faults.

Figure 4: Horizontal components of the ground acceleration used in the analyses.

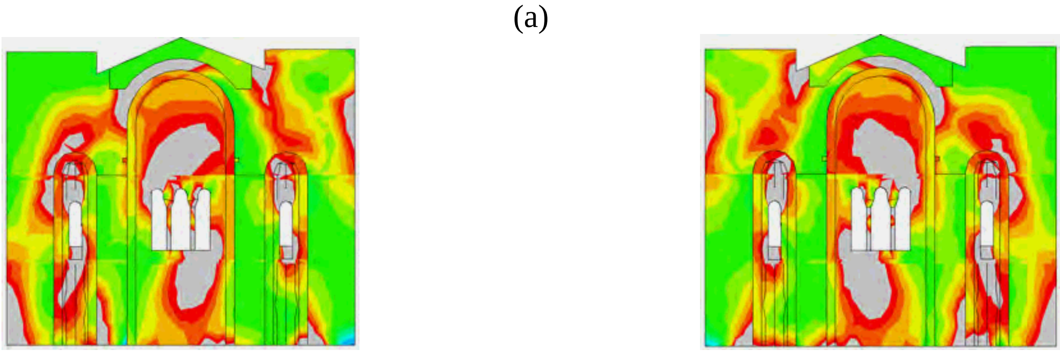
Figure 5: Corresponding elastic response spectra for 5% damping.

5. COMPARISON WITH THE EXISTING DAMAGE

From the results of the numerical analysis, the regions of the structure, at which the maximum principal tensile stress was developed for the combined effect of gravity and earthquake loads, were determined. These regions were compared to the existing crack pattern of the church. Such a comparison is shown in Figure 6 for the E wall of the main church (sanctum). In Figure 6a, the existing cracks are shown, while in Figure 6b, the principal tensile stresses are plotted for two different time instances. With gray colour are shown the areas with stresses larger than 350 KPa, a value that is significantly larger than the tensile strength of the masonry in its present condition (estimated to 150 KPa, at the most). Therefore, cracks should appear at these regions. Comparison of the gray areas of Figure 6b with the crack pattern of Figure 6a shows that the existing cracks are in agreement with the ones that would occur during the earthquake. Typical results concerning the vaulted roof are presented in Figure 7. Again, the gray areas correspond to tensile stresses larger than 350 KPa. Note that cracks do not exist at the roof of the narthex, because this part of the building was restored after the 1881 earthquake.
Similar results were obtained for the other walls of the building. In general, the areas of large tensile stresses were in accordance with the existing cracks in all walls, except for a vertical crack at the upper part of the S wall of the main church, which could not be explained. This crack, however, could be the result of

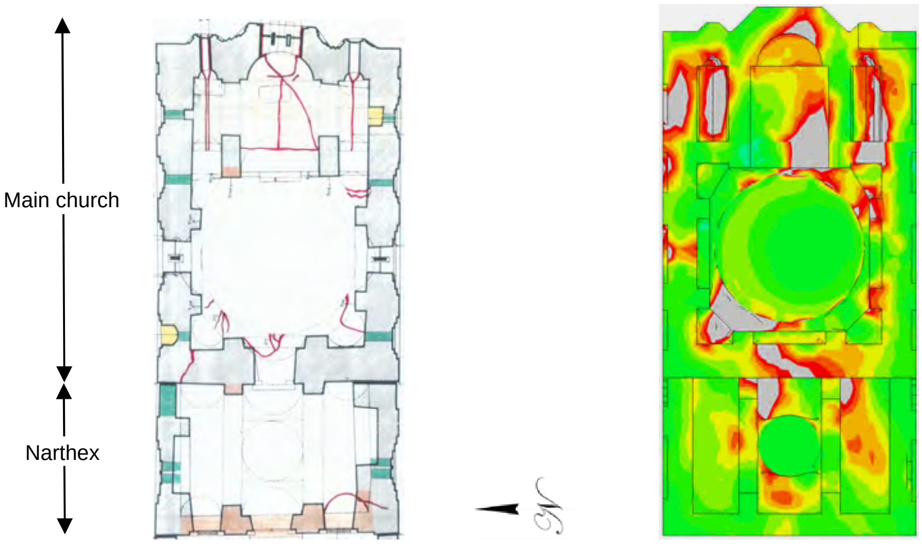


(a)



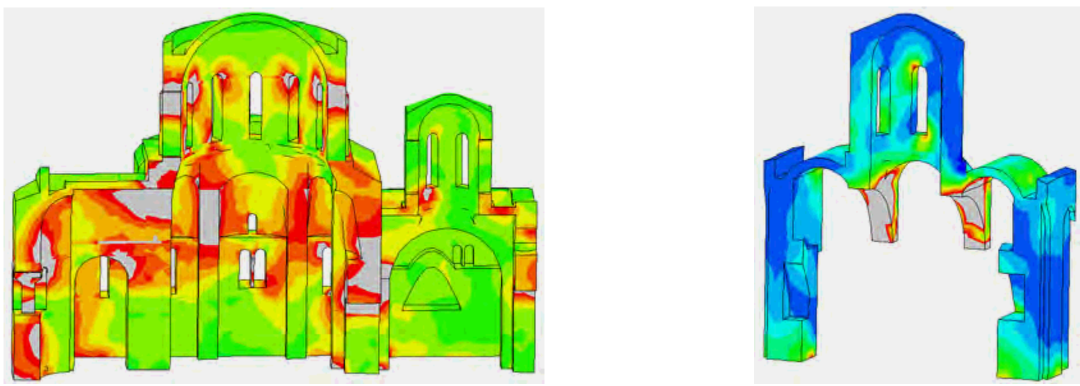
(b)

Figure 6: E wall (sanctum): (a) existing crack pattern; (b) principal tensile stresses at time steps 240 and 284.



(a) (b)

Figure 7: View of the roof (from below): (a) existing crack pattern; (b) principal tensile stresses at time step 278.



(a) (b)

Figure 8: (a) Longitudinal section of the main church and the narthex (S view); (b) Transverse section of the narthex (W view).

secondary effects due to the redistribution of stresses after cracking and/or the collapse of the main cupola during the 1881 earthquake.

It is interesting to note that the numerical results can also justify the collapse of the main cupola and the collapse of the dome and the walls of the narthex that occurred during the 1881 earthquake. For the seismic motion considered in the present analysis, Figure 8a shows that high tensile stresses develop simultaneously at the upper part of almost all the supporting columns of the main cupola; therefore, there is not enough shear strength to bear the seismic loads and the cupola may collapse during such an earthquake. In the case of the narthex, high tensile stresses are developed at the two longitudinal supporting arches (Figure

8b), which lose their capacity to bear the vaulted roof, at which high tensile stresses are also developed (not shown in the figure). It seems, therefore, that the collapse of the narthex started from the two longitudinal vaults and expanded to the vaulted roof and the walls.

6. CONCLUSIONS

In this paper, the seismic response of a Byzantine church is examined. Elastic time-history analysis was performed for a strong seismic motion and the stresses developed at the walls, the vaulted roof vaulting and the domes were derived. The areas, at which the tensile stresses significantly exceeded the corresponding strength of the masonry, were determined and compared to the existing crack pattern. This comparison showed a remarkable agreement in most cases, indicating that elastic analysis can be used to determine weak regions of the structure and the places where damage will occur during an earthquake. Therefore, such analyses can also be used to evaluate the effectiveness of strengthening measures and to provide necessary data for their design.

ACKNOWLEDGEMENTS

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THE HAMAM OF IOANNINA: ANALYSIS AND RESTORATION

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ABSTRACT

The methodology applied to the restoration of the Hamam in the city of Ioannina, Greece is presented. The monument is a typical example of Ottoman architecture of the 17th century. At present it is in ruined condition. The restoration study, based on a thorough investigation of the structural system and material properties, includes static and dynamic analyses using FEM. Dynamic analyses take into account not only the code provisions for earthquake design but also selected actual earthquake records. The results point out to the structure's vulnerable areas and give accurate account of actual damages. Moreover, dynamic analyses using selected records of major Greek earthquakes shed light to the behaviour of a complex structure under seismic loads. Overall, the methodology used could function as blueprint for interventions in similar monuments.

1. INTRODUCTION

There is increasing interest lately to establish a framework for interventions on monuments and historical structures, as well as to assess their seismic vulnerability. In a large number of reports and case studies [2, 4], design methodologies that have been applied to important monuments are presented. However there are few published reports regarding historical masonry structures in Greece given the large number and the diversity of these structures.

As the awareness of seismic hazards imposed on historical structures increases and actions for their protection become more intense, the urge to establish this framework is bigger. The study for the restoration of the Ottoman Bath in the city of Ioannina has led to a methodology that may contribute to this purpose.

2. STRUCTURE

The Hamam is a one-storey structure, having dimensions of 25.80 m x 13.30 m.

Figures 1a, 1b. Monument before and after the collapse of the soyunmalik dome.

It was constructed inside the castle of Ioannina at the beginning of the 17th century and consists of four parts [1, 6, 7] as shown on Fig. 2:

a. Soyunmalik (disrobing room): is a square room covered by a dome. The dome, constructed by bricks placed in radial configuration, sits on an octagon, visible only from the outer side. The octagon is supported by a system of squinches and pointed arch pendentives, based on the walls of the room.

b. Soğukluk (tepidarium): is a long and narrow barrel-vaulted room.

c. Sicaklik (hot area): is a square room covered by dome, having similar configuration and structural system as soyunmalik. It has three barrel-vaulted extensions, to the east, south and west. At the corners of the sicaklik four square rooms with sides of 2.40 m, are located. These are the halvet (private rooms), all covered by domes.

d. Kulhan (furnace place): is a longitudinal barrel-vaulted room at the south side of the monument, housing the furnace and the reservoir, covered partly by domes and partly by a barrel vault.

The outer shell of the structure consists of masonry made of stone and has thickness of approximately 1.00 m, while the domes, constructed with bricks, are 0.30 m thick. There is no indication of tie-rods or timber reinforcement embedded in the masonry. The floors of soğukluk, sicaklik and halvet were heated via the use of hypocaust, constructed below slabs made of marble that were placed on cylindrical stone pillars. These floors have been totally ruined. The foundation of the monument consists of strip footings (1.00 m high) made of stone.

3. PATHOLOGY

The monument has been abandoned for many years and is in a ruined condition (Figs. 1b, 2a, 2b, 3). The damages that it has endured can be attributed to weather conditions (humidity and rainfall) and earthquakes. Also, due to the complete lack of maintenance, its condition is rapidly deteriorating. After a thorough structural inspection, carried out by the authors, the pathology was recorded. Structural damages can be divided into two categories: (1) damages of general nature, concerning the disruption of continuity and shape of the structural system (partial

Figures 2a, 2b. Damages at domes and arches.

collapses, extensive cracks) and (2) localized damages concerning the deterioration of building materials (masonry, bricks, mortar).

4. METHODOLOGY OF RESTORATION STUDY

The restoration study was completed in two stages. In the first stage, the original structure was analysed in order to evaluate the safety level and determine the causes and mechanisms of structural damages. It should be noted that the term “original structure” corresponds to the monument after restoration but not reinforced (i.e. reconstruction of collapsed parts, no disruptions or cracks considered in the analysis, mechanical properties of masonry considered as measured).

In the second stage, the reinforced structure was analysed. Strengthening the monument was accomplished by improving the mechanical properties of the

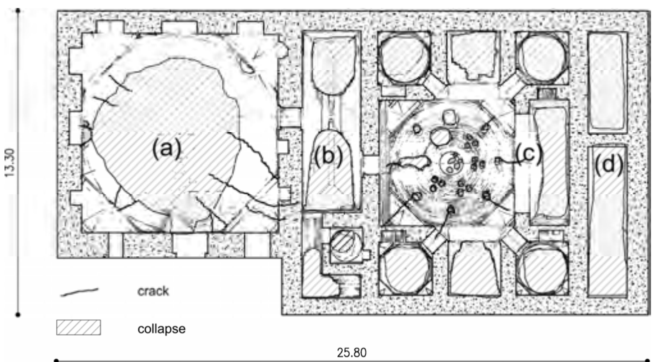


Figure 3. Ceiling plan: cracks and collapses are denoted.

masonry (by use of wall ties for the cracks, grouting, repointing of existing masonry, removal and replacement of decaying material, reinforced coatings) and the use of tie-rods. To determine the best restoration proposal, analyses were performed on a series of structural models gradually increasing the degree of intervention, by placing tie-rods. The final proposal, presented here, corresponds to the minimum possible intervention.

5. STRUCTURAL INFORMATION

This study is based on the systematic investigation of the structural system and the material properties carried out by the authors.

5.1. Material properties

The outer shell of the monument consists of stonework of limestone origin, while the domes are constructed with bricks. Stones are of small size while bricks are solid with wide surface and small thickness (3 cm). The mortar is divided in three categories based on its use (dome construction, wall construction, plumbing installations). Laboratory tests revealed the mechanical properties of materials (stone, brick, mortar) and masonry (masonry from semi-carved stones, masonry from carved stones, masonry from bricks).

Regarding the properties of stone and brick, measured quantities included bulk density, flexural and direct tensile strength. Tensile strength of mortar used in stonework and domes was also measured.

For the mechanical properties of masonry, measured quantities included bulk density, elastic constants (modulus of elasticity, shear modulus and Poisson’s ratio), compressive and tensile strength, normal and parallel to bed joints, and shear strength.

5.2. Soil

The monument rests on stiff soil, classified as category A, both from Greek Seismic Code 2000 (GSC-2000) and EC-8.

5.3. Loads

Regarding dead loads, self-weight of the structure is fairly well estimated from the data collected from the investigation of the structural system. The accuracy of the information regarding geometry (cross-sections, fillings) and materials helps to properly define actual loads. In addition to dead loads, seismic, snow and wind loads were considered. Seismic loads are described in the following section. Snow and wind loads were estimated according to the provisions of EC-1.

6. SEISMIC HAZARD

As can be derived from the available seismological data [3, 5] there was remarkable seismic activity in the region in the past. In Table 1, ample historic

Table 1. Historic earthquakes at Ioannina during the lifespan of the Ottoman Bath.

Event/Date	**Magnitude**	**Max Intensity (MM)**	Losses
1740 Ioannina	6.2	VIII (Ioannina)	Extensive building damage
1743 Corfu	7.0	VIII (Ioannina)	Serious damage in most buildings
1813 Ioannina	6.2	IX (Ekklisohori)	Minor building damage in the city, Building collapses in neighbor cities
1823 Sagiada	6.4	IX (Sagiada)	Building collapses on the axis Ioannina-Sagiada
1858 Ioannina	6.0	VIII (Ioannina)	Extensive building damage
1867 Ioannina	6.2	VIII (Ioannina)	Extensive building damage
1898 Ioannina	6.3	VIII (Ioannina)	Building collapses

evidence for earthquake-induced destructions in Ioannina during the lifespan of the monument is presented. It can be noticed that these earthquakes are important in magnitude (as estimated from correlations with intensity [3, 5]) and may have been the cause of some of the structure's damages. However on recent years seismic events in the area were moderate.

According to seismic hazard zonations of Greece, the specified design ground acceleration for the region is 0.16g. This value should be multiplied by importance factor 1.3 assigned to monument structures according to GSC-2000. To evaluate the safety factor of the structure, not only the above code provisions were considered but also actual earthquake records from major Greek earthquakes. These motions (Table 2) were selected to correspond to the seismic profile of the region regarding soil conditions (stiff soil), distance from epicenter and magnitude of event.

7. ANALYSIS PROCEDURE

7.1. Modeling

A 3-D finite element model of the structure was created (Figs. 5, 6), following the available information regarding geometry and load bearing mechanism. Computer code ETABS was used. The complexity of the structure (domes, squinches and pointed arch pendentives) has imposed the use of a large number of elements (8400 approximately). The model was restrained both vertically and horizontally at the foundation level. Analyses assume elastic behaviour. Results include normal stresses, parallel and normal to bed joints S_{11} and S_{22} respectively, shear stresses, S_{12}, S_{23}, and S_{13} and principal stresses S_{max} and S_{min}. For each element two groups of stresses were obtained: one for the inner and one for the outer side.

Table 2. Selected major Greek earthquakes recorded on stiff soil.

Event/ Date	Station	Magnitude (M_s)	Epicentral Distance (km)	Distance from Rupture (km)	Orientation		PGA (g)
Kalamata 13/09/1986	Kalamata (OTE bldg)	5.8	10	5	L	N265	0.22
					T	N355	0.30
Egion 15/06/1995	Egion (OTE bldg)	6.2	18	4	L	N-S	0.54
					T	E-W	0.49
Athens 07/09/1999	Athens (Syntagma)	5.9	17	13	L	N10	0.15
					T	N100	0.23

7.2. Response spectrum analysis

Analysis was performed following the provisions of EC-6 for the design of masonry structures and GSC-2000 (similar to EC-8). The load combinations of these codes for gravity and seismic loading were considered.

In order to further investigate the behaviour of the monument and check the performance of the reinforced structure under selected earthquake motions of Greek territory (Table 2), the corresponding response spectra in both horizontal directions were considered in the analysis. In Fig. 4, the response spectrum of the dominant direction of each motion is plotted and compared with that suggested by GSC-2000. It should be noted that the behaviour factor of the structure was taken as 1.0, i.e. no ductility or overstrength were considered.

8. STRUCTURAL ANALYSIS OF THE ORIGINAL STRUCTURE

In the first stage, the original structure, corresponding to the monument after restoration but without strengthening, was analysed. The fundamental period of the monument is approximately 0.05s, in both the longitudinal and transverse axis verifying the high stiffness of the structural system. Examining the results, it can

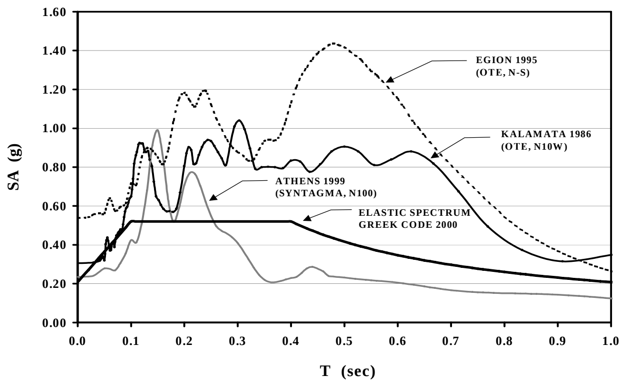


Figure 4. Acceleration response spectra (ζ=5%).

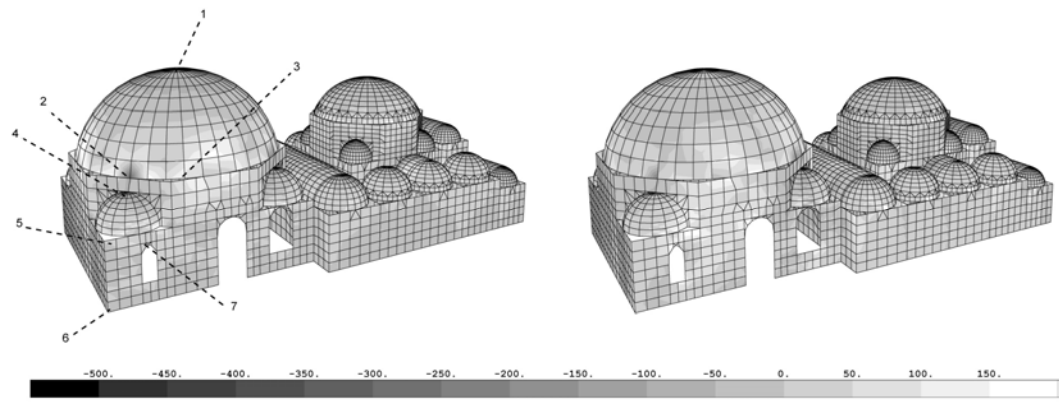
Table 3. Comparison of normal stresses (S_{11}, S_{22}) at selected points from dynamic analysis using the GSC-2000 elastic response spectrum.

Position	**Original Structure**				**Reinforced Structure**			
	S_{11} (KPa)		**S_{22} (KPa)**		**S_{11} (KPa)**		**S_{22} (KPa)**	
	inside	**outside**	**inside**	**outside**	**inside**	**outside**	**inside**	**outside**
1	-30.0	-60.6	-29.3	-60.3	-25.9	-56.0	-29.2	-60.2
2	52.1	-310.3	-82.1	-393.0	-119.0	-217.1	-45.1	-308.2
3	-89.2	65.9	18.4	-18.4	-99.1	-17.3	10.1	-23.4
4	61.1	-133.2	62.2	-97.9	48.2	-129.9	55.9	-82.7
5	-45.4	-50.7	-30.8	-5.0	-20.3	-116.0	-24.8	-7.9
6	71.0	149.1	88.1	60.2	75.1	151.2	95.3	63.8
7	6.1	11.2	47.9	29.1	8.1	12.8	53.1	36.3

be concluded that compressive stresses are small compared to the strength of masonry. This is not the case with tensile stresses that exceed the capacity of the structure at several points, especially on the upper part of the large domes of soğukluk and sicaklik. On Figs. 5a and 5b, the vulnerable areas of the structure are denoted. In Table 3, normal stresses, parallel and normal to bed joints, S_{11} and S_{22} respectively, on selected points of the structure, denoted on Fig. 5a, are presented. As can be derived by comparing Figs. 5a and 5b to Fig. 3 the results reveal the weak areas of the monument and give an accurate account of the actual partial collapses and cracks. This shows the validity of the model used, crucial to continue to the next stage and also proves that despite the growing use of inelastic methods, elastic analysis can still be a sufficient tool.

9. STRUCTURAL ANALYSIS OF THE REINFORCED STRUCTURE

In the second stage, the reinforced structure was analysed. Strengthening the monument was accomplished by improving the mechanical properties of the masonry



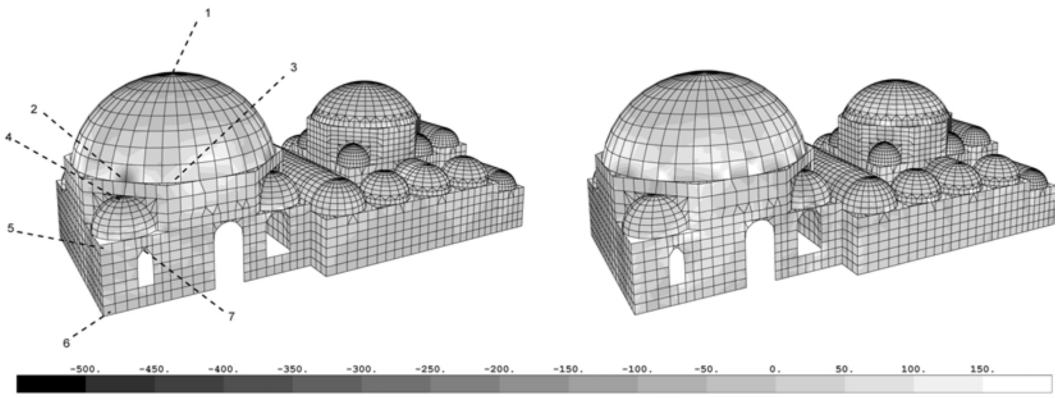
Figures 5a, 5b. Original Structure: maximum principal stresses (KPa) from gravity and seismic loads respectively.

and using tie-rods. The latter, after taking into account the results of the analysis of the original structure, are placed on the octagonal base of the major domes and on the crown of the perimeter walls. To determine the best restoration proposal, analyses were performed on a series of structural models, gradually increasing the degree of intervention, by adding tie-rods. The final proposal, presented here, corresponds to the minimum possible intervention. The reinforcement does not affect noticeably neither the stiffness nor the mass of the structure, so the fundamental period is the same as in the previous analysis. Principal stresses S_{max} are presented on Figs. 6a and 6b for gravity and seismic load combinations respectively. In Table 3, normal stresses, parallel and normal to bed joints, S_{11} and S_{22} respectively, on selected points of the structure, denoted on Fig. 6a, are presented. It can be concluded that tensile stresses, still located in the same areas of the monument as in previous analysis, have decreased by 10% approximately.

In order to further investigate the behaviour of the monument and check the performance of the reinforced structure, analyses under selected motions of Greek territory (Table 2), were carried out. These motions impose greater actions than the provisions of GSC-2000. Response spectrum analysis using the records of Athens (1999) and Kalamata (1986) earthquakes result in tensile stresses that slightly exceed the strength of the masonry. This is not the case with the severe Egion (1995) earthquake; tensile strength of masonry is surpassed and therefore inelastic analysis would be needed as a further step to predict the monument' s behaviour.

10. CONCLUSIONS

The methodology followed for the restoration of the Ottoman Bath in the city of Ioannina is presented. The restoration, based on a thorough investigation of the structural system and the material properties, includes static and dynamic analyses using FEM. Dynamic analyses take into account not only the code provisions for earthquake design but also selected actual Greek earthquake records.



Figures 6a, 6b. Reinforced structure: maximum principal stresses, (KPa), from gravity and seismic loads respectively.

The study consists of two stages. In the first, the original (unreinforced) structure was analysed and the results point out τo the structure's vulnerable areas and give accurate account of the actual partial collapses and cracks. It is concluded that despite the growing use of inelastic methods, elastic analysis can still be a sufficient tool in predicting the behaviour of a complex monument. In the second stage, analyses were performed on a series of structural models gradually increasing the degree of intervention to determine the best proposal (minimum intervention). Comparison of results between the analyses of the final model to the ones of the original structure verifies the correctness of suggested interventions. Moreover, dynamic analyses using selected records of major Greek earthquakes shed light on the behaviour of a complex monument structure under actual seismic loads. Overall, the methodology used, could function as blueprint for the restoration of similar monuments.

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CHAPTER VI

Intervention, Restoration and Preservation Techniques and Methodology

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PROPOSAL OF A NEW BASE-ISOLATION SYSTEM FOR HISTORIC TIMBER STRUCTURES FOR THE SAFETY OF THEIR STRUCTURAL AND NON-STRUCTURAL COMPONENTS

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ABSTRACT

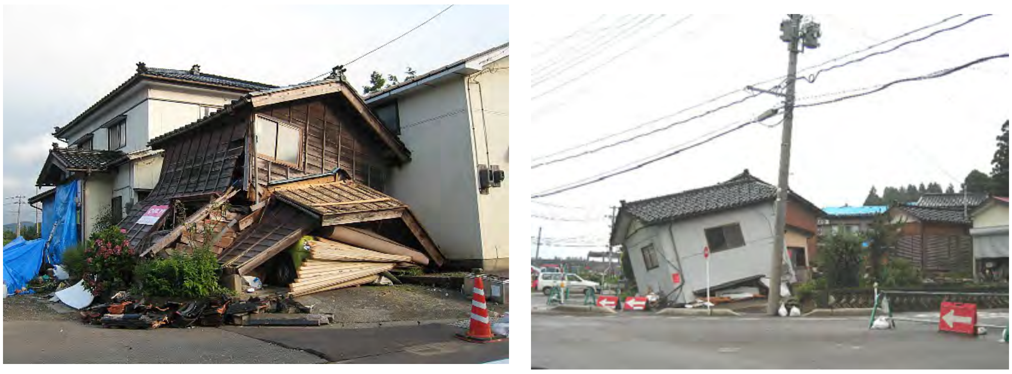
For the preservation of historical buildings safety of both structural components and non-structural component are important. A new base-isolation device was proposed for the relief of the seismic load to these components. The results of full-scale shaking table test using a real scale wooden house are reported.

1. INTRODUCTION

Earthquake is the critical problem for every structure constructed in seismic regions (figs.1 and 2). In order to avoid fatal collapses of the structure against the heavy inertia force under the excitation of ground motion during earthquakes structures in seismic regions are usually constructed with stronger skeleton and old existent structures are reinforced to be "earthquake resistant". However since the purpose of the "earthquake resistant structure" is "not to collapse" it never relieves the seismic load. Therefore "non-earthquake resistant" parts, especially "non-structural components", are exposed to the seismic load and deformation that is induced during earthquake (fig.3). Damage to non-structural components are sometimes very dangerous to the people using the structures and when the structures are of the historical importance it may cause serious damage to the historical heritages preserved in and around the structures. In 1997 the famous wall paintings in the church in Assisi were damaged by an earthquake although the church itself survived.

(a) A historical timber temple (b) An old church

Figure 1: Historical structures collapsed by Kobe Earthquake in January, 1995



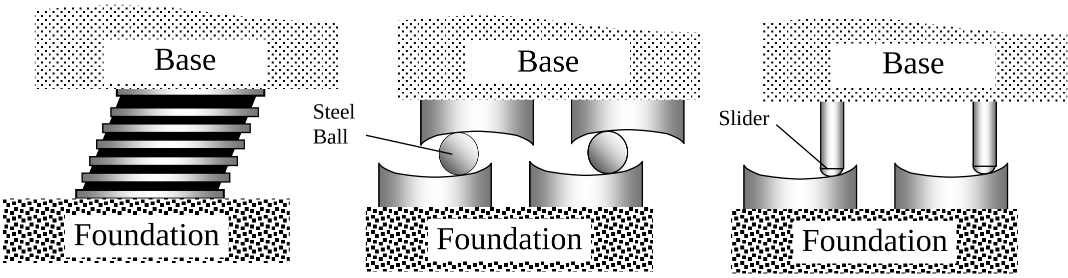
(a) A Wooden house 1 (b) A Wooden house 2

Figure 2: Wooden houses collapsed by Chuetsu-Oki Earthquake in July, 2007

(a) Collapse of ceilings 1 (b) Collapse of ceilings 2

Figure 3: Non-structural components damaged by earthquakes

Base-isolation systems are the structural systems that isolate the super structures, using special bearing devices (fig.4), from the ground motion during earthquakes. Base-isolation systems make it possible to drastically reduce the seismic loads and deformations induced to both the structural components and non-structural components. In this paper a new base-isolation system for timber structures is proposed and the results of its real scale shaking table tests are reported.

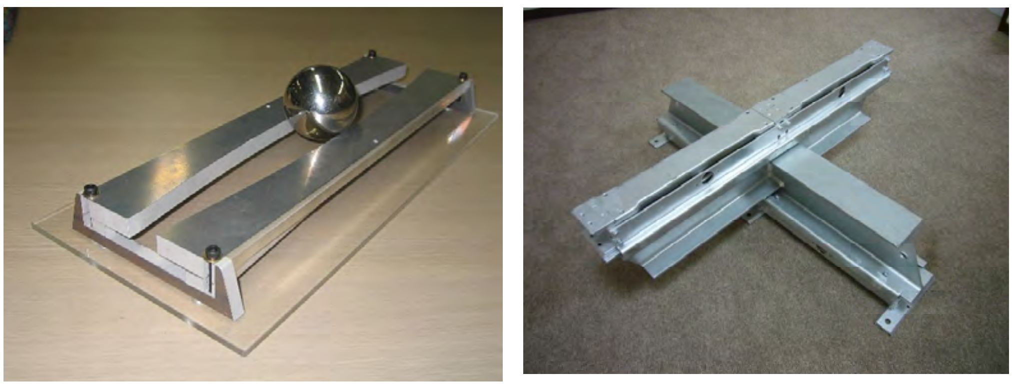


(a) Rubber Bearing (b) Friction Pendulum 1 (c) Friction Pendulum 2

Figure 4: Typical Base-isolation devices with recovering mechanisms

2. A NEW BASE ISOLATION SYSTEM

The base-isolation system proposed is based on the new concept of pendulum, the gage pendulum (fig.5(a)), of which recovering force generated by the gravity force and the shape of the gap between a pair of curved lines. This system does not require any slope for the ball to roll to produce the pendulum movement. Using this concept the final configuration of the base-isolation device became cross rail style as shown in fig.5 (b) and was named “VP-isolator”.



(a) Gage pendulum (b) VP-isolator

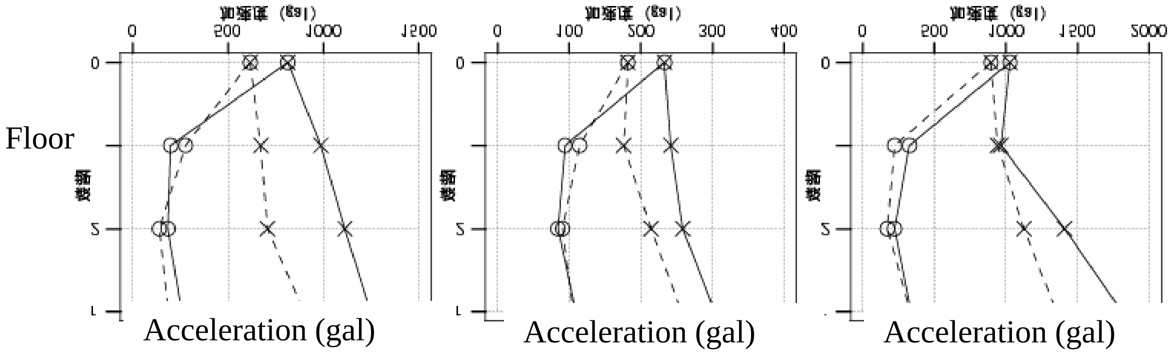
Figure 5: The new base isolation device

3. FULL SCALE SHAKING TABLE TEST

In order to check the performance of this base-isolation system a series of full scale shaking table test using real wooden house was carried out. The size of the base of the house was 6.37(m) x 7.28(m) = 46.37 (m^2) and the height of the house was 7.4m. Total weight of the house was 240kN. Six VP-isolators and four hydraulic dumpers are installed and connected between the base and the foundation of the house. After the tests of base-isolated house the same house was directly connected to the foundation without base-isolation system and another series of shaking table tests were carried out to investigate the behavior of "earthquake resistant" house.

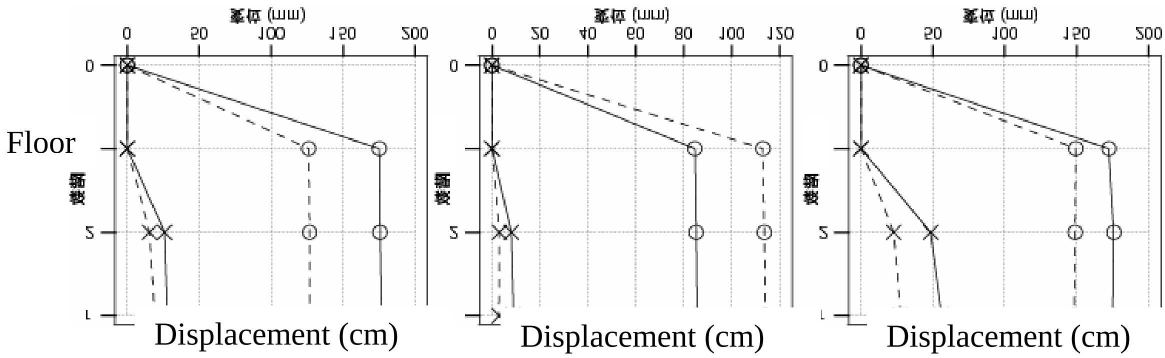
Figure 6: General view of the full scale shaking table test.

Several real earthquake ground motions recorded, Kobe 1995 (max. 818gal), Hachinohe 1968 (max. 233gal) and Ojiya 2004 80%(max. 1046gal), were reproduced by the shaking table. Some results of the tests are shown in figs.7 and 8. In the figs. the crosses show the results of base-isolated house and the circles show the results of the earthquake resistant house. With base-isolation system acceleration level was reduced to about one-fifth of the that of earthquake resistant house. Contrarily, relative displacement between the base and the foundation became much larger. This large displacement absorbed the large ground motions. For the base-isolated house structural components and non-structural components remained perfectly safe while in the earthquake resistant house structural components, especially timber braces, and non-structural components, such as wall panels and furniture were heavily damaged.



(a) Kobe (b) Hachinohe (c) Ojiya (80%)

Figure 7: Response acceleration



(a) Kobe (b) Hachinohe (c) Ojiya (80%)

Figure 8: Response relative displacement

4. CONCLUSIONS

A new base-isolation system based on the new concept of gage pendulum was proposed and developed. Through the full-scale shaking table test the high performance of the system was proved. Both structural and non-structural components were perfectly guarded by the base-isolation system.

ACKNOWLEDGEMENT

The author would like to acknowledge the support of the Ministry of land, infrastructure and transport of Japan to our research on the safety of non-structural components.

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STRENGTHENING OF MASONRY VAULTS AND INTERACTION OF VAULTS WITH OVERBURDEN

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ABSTRACT

Performed experiments on additionally strengthened vaults with metallic helical reinforcement and nonmetallic composite glass reinforcement (GFRP) proved expressive influence on carrying - capacity of masonry vaults. These experiments were nevertheless carrying out on vaults without overburden. In which way then will take effect interaction of vault, nonstrengthened or strengthened, with overburden? Realization of experiments in real conditions would be technically and financially very difficult, and therefore are vaults in combination with overburden simulated in computation programme Atena on the basis of earlier performed experiments. In addition will be performed also dynamic test of strengthened vaults with GFRP.

1. INTRODUCTION

Masonry continues to be popular because of its relative simplicity of application in the technical practice. Indeed, for a new use of structural masonry reasonable constructional rules are required, because conventional approach based on the experience is unacceptable nowadays. In addition, most methods of carrying capacity assessment and of strengthening for the existing masonry construction are increasingly based on analyses of mathematical simulation and appropriate (linear and nonlinear) computational models. One method of load-bearing elements strengthening is application of additional external reinforcement into chases in masonry on bottom side of vaults, which will provide stiffening and increasing of load carrying capacity of the individual load-bearing elements. This paper is based on the experiments in the field of masonry structures strengthening that were performed on Faculty of Civil Engineering Brno University of Technology.

In this paper are presented the results of the load tests of masonry vaults strengthened with the metallic helical reinforcement system Helifix and with non-metallic glass reinforcement (GFRP) (Figure 1). The aim of this work is to document possibilities of the use of the additional reinforcement for the strengthening of masonry structures loaded with the interaction of a normal force and a bending moment and to verify experimentally the behaviour of specially shaped profiles of the HeliBar reinforcement and the HeliBond grout in masonry, respective glass reinforcement (GFRP) and the Sikadur grout.

The method of additionally inserted non-prestressed reinforcement allows additional strengthening of masonry structures without a necessity of large intervention into vaults especially in case of external application. This system is capable redistributing newly originated stresses from load that act on a strengthened construction. The aim of reinforcement is to restrict the development of existing cracks and eliminate possibly an origin of the new ones, and to improve load-bearing capacity of vaulted masonry constructions.

Figure 1 Shape of Helibar and wrapped surface GFRP

2. DESCRIPTION OF VAULTS

Within experimental parts of the project three sets of masonry vaults with for various loading types were manufactured. For the distinction of individual vaults are used notation jKi, where „j" corresponds to series number (1-3) and „i" to the strengthening method (1-3). The vaults were symmetrically loaded in ½ of the span - 1.series (j=1), asymmetrically in ¼ of the span - 2.series and symmetrically in both quarters of the span - 3.series (j=3) (Figure 2). Each series consists of three vaults: non-strengthened one – comparative (i=1), a vault reinforced in two chases (i=2) and a vault reinforced in three chases (i=3). The vaults were bricked up from full burnt bricks on lime-cement mortar of the width 890 mm, span 2600 mm, deflection 750 mm and radius 1500 mm. Into every reinforcing chases were embedded 2 bars. Previous experiments were performed with reinforcement HeliBar of special helical shape of diameter 8 mm. For verification of behaviour on another's type of reinforcement was selected glass armature of diameter 6 mm and used only unsymmetrical loading in ¼ of the span (2.series) [1,2,3].

Last series of vaults were loaded by dynamical loading and strengthened with glass reinforcement and load only asymmetrically in ¼ of the span too.

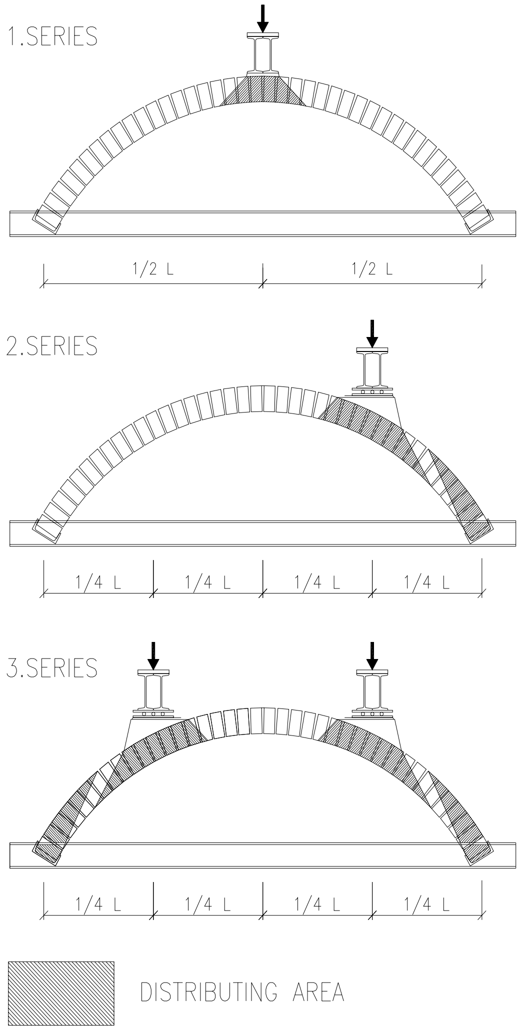

Figure 2 Loading schemes of vaults and distribution of load in vaults

3. INTERPRETATION OF TEST RESULTS

From the comparison of the load-bearing capacity of the individual vaults in the series results that essential growth of the load-bearing capacity was achieved especially in the case of 1st series and 2nd series of the vaults, namely more than eight multiple growth. This growing of carrying-capacity can be watch for both cases of reinforcement – helical metallic and glass nonmetallic. It was related to the vaults stressed by either concentrated or one-sided load, at which the vaults were loaded by the interaction of normal forces and bending moments. That's why was on basis of previous experiments [1,2] select unsymmetrical loading in ¼ of the span for vaults strengthened with glass reinforcement (Figure 3). In the case of 3rd series the experiments did not prove the effects of strengthening by additionally inserted reinforcement on the vaults load-bearing capacity; no effects of reinforcement demonstrated themselves because the vaults were mainly compressed. The result values of the loading and corresponding deformations for all series of vaults strengthened with metallic reinforcement are presented in [1,2,3].

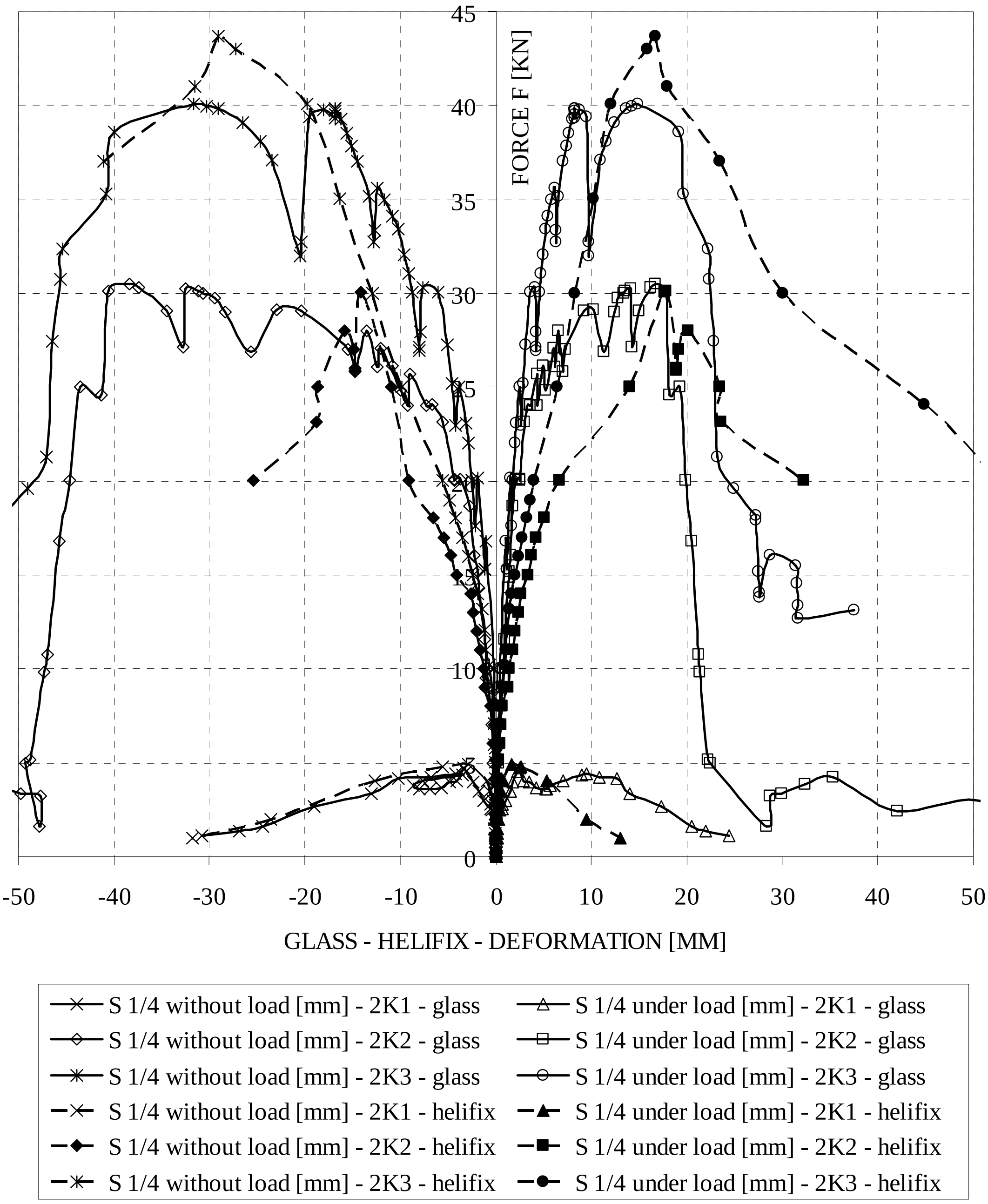

Figure 3 Comparison of deformations on vaults loaded in ¼ of the span strengthened with GFRP and metallic helical reinforcement – statical test

4. INTERPRETATION OF DYNAMICAL TESTS

Dynamical tests were performed on vaults loaded asymmetrically in 1/4 of the span and reinforced with glass reinforcement (GFRP). From results of first dynamical tests is again visible increasing of load-bearing capacity of reinforced vaults (2K2, 2K3) compared to vault unreinforced (2K1) (Figure 4). But low set of tested specimen prohibited comparison with test data from statical experiments and it also in connection with big nonhomogeneity of masonry constructions. As well a fracture mode, failure of vault by opening of tension cracks in bed joint, is not uniform and position of crack can influence final load bearing capacity. Especially load-bearing capacity of unreinforced vault loaded by dynamical loading is higher in comparison with statical examination. Strengthened vaults can be partially compared by relation of their load-bearing capacity. Ratio of load-bearing capacity of vaults with three reinforcing chases and with two chases (2K3/2K2) at statical examination is 1,33 and at dynamical examination is 1,29.

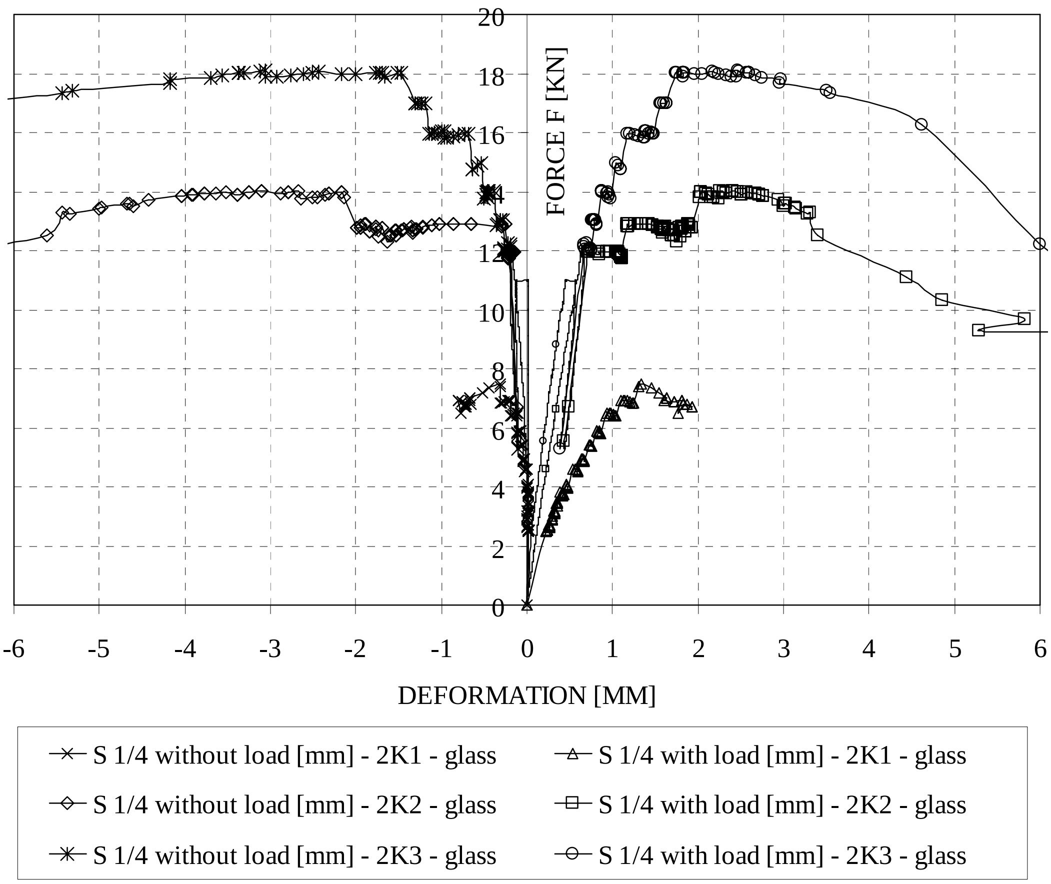

Figure 4 Comparison of deformations on vaults loaded in ¼ of the span strengthened with GFRP reinforcement – dynamical test

5. MATEMATICAL MODEL IN PROGRAM ATENA

From the theoretical facts result that the most convenient model for describing orthotropic non-continuous character of masonry is a micro-model. This model can describe not only the materials characteristic of individual materials (bricks, mortar), but also their co-acting that is in the mathematical model of masonry considered by 2D contact among the materials. This contact task describes in the best way the behaviour of masonry on the boundary of the masonry units and mortar. A disadvantage of the micro-modelling is its high time-consuming of computation and extensive number of the physically-mechanical properties to be determined for the material behaviour description and for the contact behaviour description among individual materials.

Appearance to shapes variety of vaulted masonry construction is this way optimal for investigation of these structures. Mathematical simulation is also suitable for investigation of overburden influence on load-bearing capacity of strengthened masonry vaults (Figure 5, Figure 6).

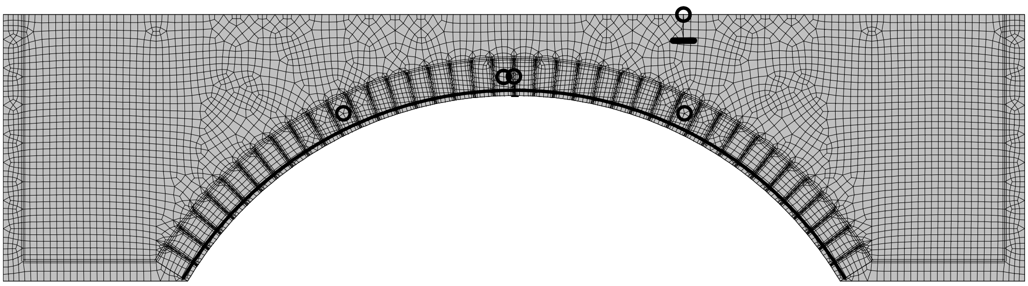

Figure 5 Mathematical detailed micro-model of masonry vault with overburden – simulation of tested shape of vault (span 2,6 m, deflection 0,75 m)

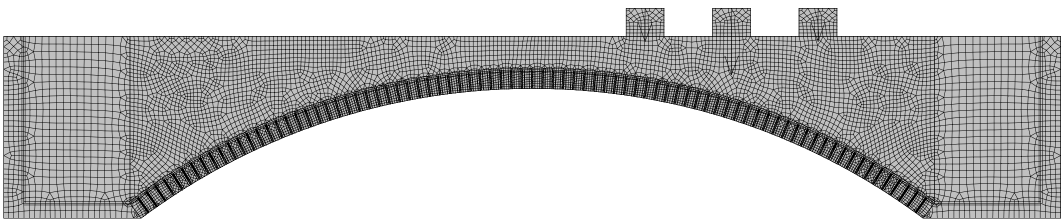

Figure 6 Mathematical detailed micro-model of masonry vault with overburden – simulation of real construction (span 6 m, deflection 1 m)

6. CONCLUSIONS

The method of repairs and strengthening of the vaulted bridges using additionally inserted reinforcement has a wide usage. Its application is possible in the cases when in a structure either originates or may originate the tension stresses in unreinforced masonry, whose magnitude is close (or exceeds) to the strength of unreinforced masonry, i.e. in places where the cracks on a construction have been already developed, alternatively when their origin is expected, whereas it may dealt with the strength of masonry in plain tension, in tension in bending or in main tension.

In the next phase of investigation we will concentrate on dynamic examination of vaults and mathematical simulation of reinforced and unreinforced vaults in interaction with overburden, which would had prove influence of reinforcing system on carrying - capacity of whole construction (vault/overburden). For comparison of overburden interaction with masonry vault and influence of strengthening will be except experimentally tested vaults also simulated vaults with other shape and proportions.

ACKNOWLEDGEMENT

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BASE ISOLATION IN THE RETROFIT OF MASONRY CHURCH BUILDINGS

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ABSTRACT

The severe damage of masonry buildings after earthquakes has pointed out the need of studying new seismic protection techniques in order to guarantee appropriate safety level against earthquakes. In historical buildings, like basilica churches, a seismic rehabilitation should have also a minimum impact on the construction, preserving the original structure as far as possible. In the light of a previous study conducted by the authors, in this paper an improved optimization process has been applied on some study cases. The attention has been focused to assessing the differences between two configurations, with and without rigid diaphragm connecting the isolators. The analysis results show that the presence/absence of the rigid floor does not imply variations in terms of accelerations, displacements and forces in the upper part of the construction, but only different values of the normal stress on the isolators for vertical load, due do the different deformability of the upper-structure.

1. INTRODUCTION

Base Isolation System (BIS), widely used for seismic protection of new-designed buildings in earthquake prone countries, today is increasingly being used also for the seismic retrofit of existing structures.
Remarkable examples of seismic upgrading of historic buildings through BIS, realized in USA, between the end of the 80s and the 90s [1, 2], have shown that the feasibility of the intervention has to be examined under many aspects which are strictly related to the architectonical, structural and historical characteristics of the construction. Some specific design and construction issues which are related to the application of BIS to historic buildings are: the seismic safety level to be adopted for the upgraded building; the comparison among the BIS solution and other conventional retrofit schemes and the reasons underlying the choice of BIS;

the interaction between the strengthening interventions in the upper part and at the foundation level; the sequence of the constructional phases, including the installation (placement and jacketing) of the isolation devices and the definition of the structural details necessary to this purpose.

With specific reference to the churches, some additional issues have to be considered in the feasibility study: above all, the existence of underground volumes, such as crypts, as well as of adjacent constructions, are factors which could make unfeasible such a retrofit strategy. Further, the presence of valuable and precious floor at the base of the structure can undermine a plain application of base isolation, since in current design practice a r.c. slab at the isolation level is introduced. Therefore it seems particularly important to assess the effect of either inserting or not a rigid floor connecting the isolators, at the base of the structure.

In this paper two church case studies are considered for discussing some specific design problems which came into picture in the retrofit through BIS; in particular the design procedure for the isolation system is described and applied to the two case studies; then, the dynamic performance of the structures is assessed both in terms of global behaviour and in terms of stress distribution among isolators.

2. ISOLATION SYSTEM DESIGN

It is well known that in the design process of an isolation system, the best balance between two conflicting requirements, i.e. maximum filtering of the ground motion and acceptable displacements at the structure base, must be achieved. For this aim, target isolation periods in the range of 2.5-3 s, and damping values (expressed in terms of equivalent damping ratio ξ) between 10% and 20%, are usually selected. However a number of additional design parameters have to be properly adjusted in order to gain also a satisfactory performance both of the global isolated structure and of the single isolator units. In the following, High Damping Rubber Bearings (HDRBs), which combine the isolation, damping and load-support functions, are considered as isolation units.

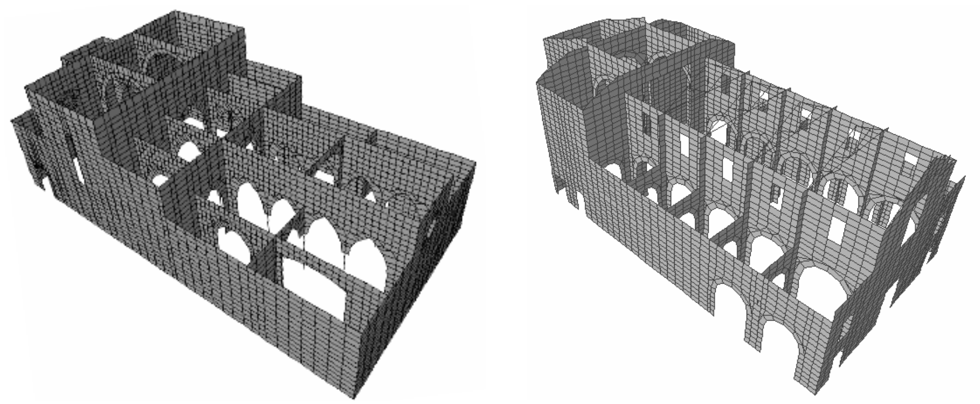
The basic input of the design process is the upper-structure layout, i.e. mass and plan position of centroid G_{mass}; the locations of isolators are grossly defined at the intersections of transversal and longitudinal walls in the church base plan; consequently, the vertical load N_i acting on each isolator is evaluated.

Tentative values of the target isolation period T_{is}, and of the design vertical stress σ_v for the devices, are set respectively equal to 3 s and 6 MPa. The stiffness of the isolation system K_{tot} can be derived by adopting a SDOF approximation, while the diameters of the isolators D_i can be derived as a function of the design vertical stress. Selecting a rubber compound, the value of the shear modulus G is fixed; (soft rubber, G=0.4 MPa, normal rubber G=0.8 MPa and hard rubber G=1.4 MPa). By fixing an unique value of the first shape factor S_1 (equal to 20) for all isolators and of the second shape factor S_2 (equal to 3), the steel thicknesses t^i_i and t^i_r are computed for the various diameters. However, homogenization of the device total height is necessary for simplifying construction aspects; thus by homogenizing the total heights of the isolators, minimum thickness values of steel shims and rubber layers are obtained and adopted for all devices to respect the limitations for S_1 and S_2 ($S_1 \geq 20$; $S_2 \geq 3$).

The effective values of S_1, S_2, K_{Hisol}, K_{Htot}, T_{is} and the position of the stiffness centroid C_{stiff} are finally computed and the eccentricity between the mass and the stiffness centroids can be evaluate; if the eccentricity is significant, the values of G, T_{is} or σ_v need to be adjusted, otherwise the procedure ends and the specific checks on the isolators, as prescribed by the code provisions, can be performed.

3. CHURCH CASE STUDIES

The buildings considered in this study are the "S. Ippolisto" (SI) and "S. Giovanni a mare" (SGMR) churches, both located in Southern Italy. In Fig. 1.a and 1.b, the 3D FE models of the two buildings are reported.



(a) (b)

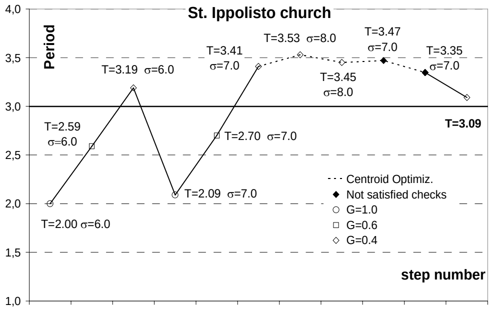
Figure 1. Study cases: a) SGMR; b) SI: 3D models.

For the two churches, an isolation system, consisting of HDRBs and sliders, has been designed in order to shift the structure period to 3 s; the iterative procedure has been applied until the isolation period was equal the to target value and the mass and stiffness centroids were almost coincident (Fig. 2).

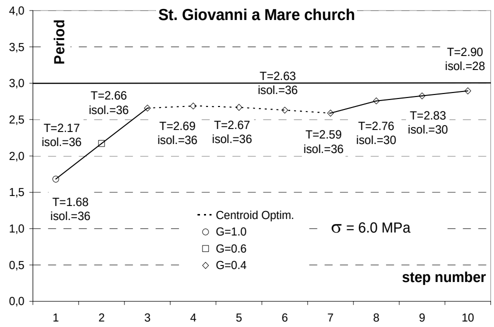
The design procedure has shown different crucial points in the application to the two case studies, characterised by a quite different weight; for the SI church, with a weight of 54830 kN, the isolation target period has been easily achieved, whereas the check for critical load of the single isolators has been the main problem; on the contrary, in the SGMR church, with a much smaller weight (36850kN), all isolator checks were easily satisfied, but the isolation target period was really hard to get, also with small rubber shear modulus (G=0.4MPa).

For this reason sliding devices have been placed under some interior walls of the church; the multidirectional teflon sliding devices, constituted by a stainless steel plate fixed to the superstructure and a teflon disk fixed to the substructure, allow any relative translation among the superstructure and the substructure. These devices only carry the vertical loads and do not add lateral stiffness, thus not affecting the isolation period.

These considerations are better shown in Figure 2.a and 2.b, where the isolation period is plotted vs. the design iteration number; each step is characterised by a design vertical stress σ, a value of the isolation period T_{is}, a value of the shear module G, and a plan configuration of HDRBs. The target T_{is} equal to 3s (horizontal line) has been traced as the reference value.

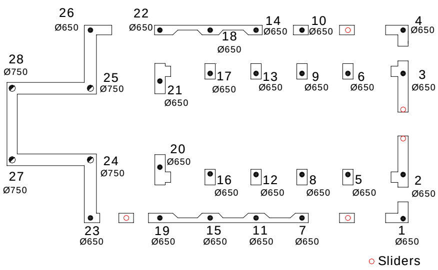


(a)

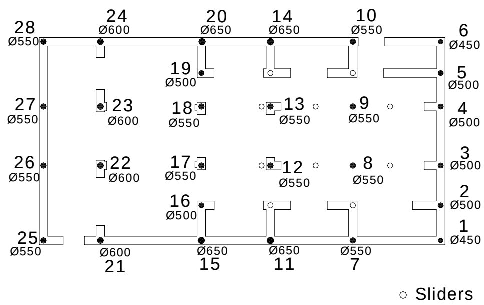


(b)

Figure 2. Application of the design procedure: iterations: a) SI; b) SGMR.



(a) SI – Step 11



(b) SGMR – Step 10

Figure 3. Placing of the isolators and sliders: a) SI; b) SGMR.

The final HDRBs configuration in the SI church (Fig. 3.a) consists of 28 isolators and 5 sliders. It was obtained an isolation period of 3.09 s, with devices Ø 650 (24 isolators) and Ø 750 (4 isolators); for both isolator diameters: single rubber layer thickness ti is equal to 6.5 mm; total rubber thickness tr is equal to 169 mm.

In the isolation system of the SGMR church (Fig. 3.b), 28 HDRBs and 10 sliders have been used. The isolation period is 2.90 s and the HDRB diameters are: Ø 450 (2 isolators); Ø 500 (6 isolators); Ø 550 (12 isolators); Ø 600 (4 isolators); Ø 650 (4 isolators). Single and total rubber thickness are respectively t_i=5.5 mm and t_r=154 mm. In Table 1 the checks for the isolators in the SI churches are reported, according to the italian seismic code [3]. The checks concern: maximum shear deformation (1), elastomer-steel bonding (2), stresses in the steel plates (3), Critical load (4) and Rollout (5).

Table 1: Checks on SI isolators.

No.	D [mm]	Max shear strain (1)				Bonding (2)	Steel stresses (3) [MPa]	Critical load (4) [kN]			Rollout (5) [mm]	
		γ_c	γ_s	γ_a	γ_t < 5	γ_s < 2	σ_s < 275	$V_{cr.}$	V	V_{cr}/V > 2	d_{Emax} $d_{Emax} < 1.5d_E$	$1.5d_E$
3	650	2.42	1.33	0.0043	3.75	1.33	136,1	6.9E+03	3.0E+03	2.32	231.9	614.7
24	750	1.95	1.27	0.0031	3.22	1.27	126,8	1.4E+04	4.2E+03	3.31	220.5	711.4

4. EFFECT OF THE RIGID FLOOR AT THE ISOLATION PLAN

In this section, the behaviour of the isolated churches in presence or absence of the rigid diaphragm at the isolation level is discussed in terms of local isolator demands (stress distribution among isolators).

The analyses carried out for the two churches (design spectrum analyses) have shown no remarkable differences in terms of global behaviour between the models with/without rigid floor. As expected, the fundamental period of the structure slightly decreases as the rigid slab at the isolation level is introduced; in this case, also larger mass participating factors than in the case of absent rigid floor have been obtained. No coupling effect arise as a consequence of the absence of rigid floor at isolation level and almost no difference has been detected in the displacements in the two models.

The distribution of the compression stress on the devices are almost similar in the models with or without rigid slab, but in some cases the stress values are quite different; in particular, as shown in Figure 4, the differences between stress values in the two models (with/without rigid floor) occur (1) due to the only vertical load condition, and (2) for isolators placed under masonry macro-elements characterized by large opening ratio. In Figure 5 the isolators where the maximum differences between stress values in the two models are shown together with the relevant macroelements.

In fact, the rigid floor at the isolation level acts as a rigid tie applied at the base of the macroelement opening. This effect can be clearly explained through the simplified models of Figure 6.a and 6.b, representing a single portal frame with two bearings under each pier, (a) with, (b) without rigid slab at the base: it can be observed that the rigid slab does not allow the relative movement of the pier bases, giving rise to a bending moment reaction which is transferred to the bearing couple, located under each pier, as axial load. In absence of rigid slab, the reaction at the pier base, and consequently the axial loads transferred to the bearings, assume different values.

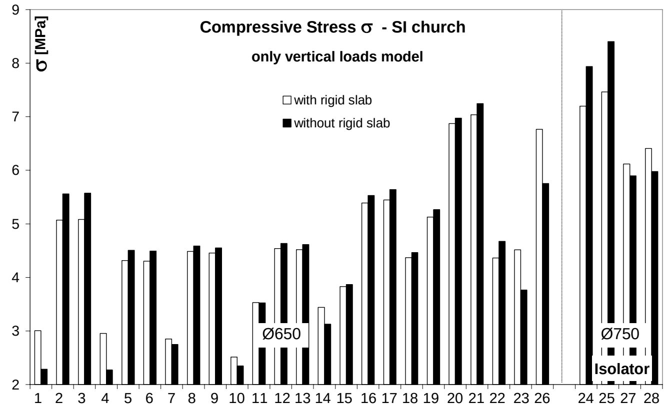


Figure 4. Compressive stresses due to vertical load.

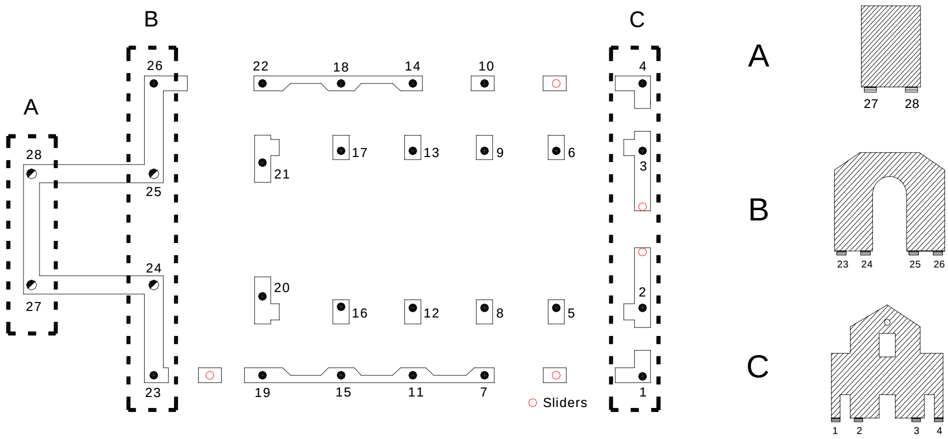
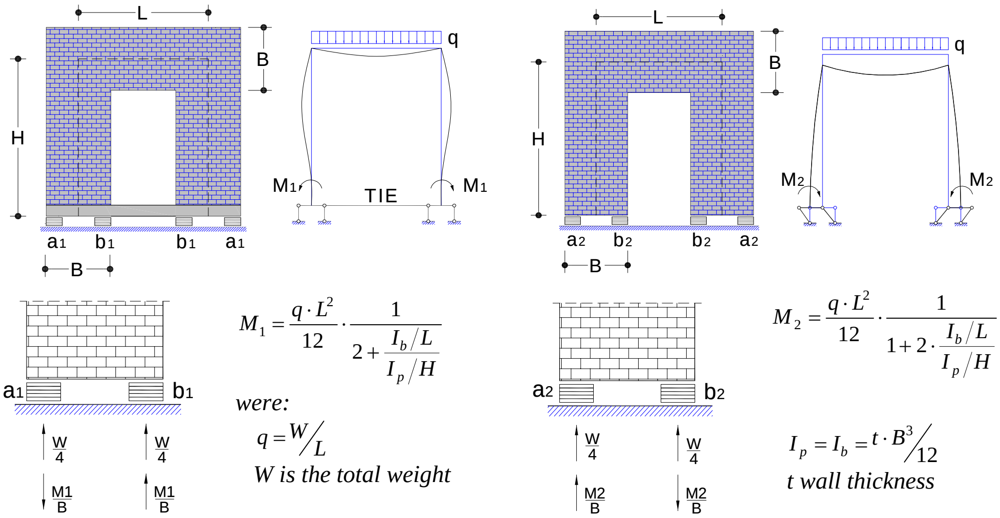


Figure 5. Macroelements in SI church - major scatters between vertical stresses.

In order to assess the influence of the deformability of the upper-structure according to the presence of a rigid slab at the base or not, a parametric analysis has been carried out on the simplified models. Different values of the opening ratio (i.e. the ratio of the opening width (L-B) to the piers width (2B)) have been considered with a stepwise variability of 0.5, 0.75, 1 and 1.25. In Figure 7 the ratio between the values of axial load in the two models (with/without rigid tie), acting on the couple of isolators a and b, located under a pier, is provided as a function of the portal opening ratio. It can be observed that the variation can reach values greater than 30%.



(a) (b)

Figure 6. Simplified models: portal (a) with rigid tie, (b) without rigid tie.

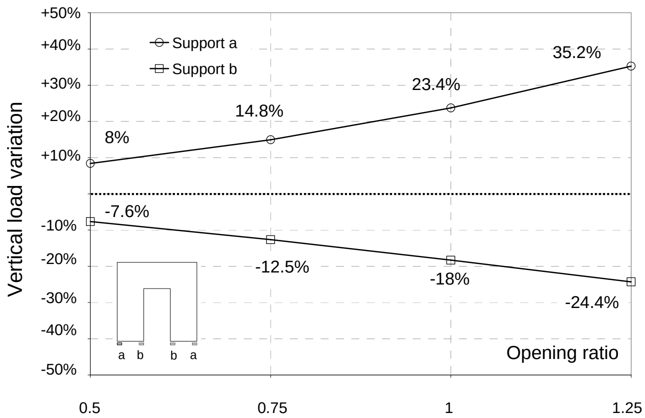


Figure 7. Effect of the deformation in the upper-structure.

The macroelement B of the SI church, represented in Figure 5, is quite similar to the simple portal model employed in the parametric analysis; the macroelement B is characterized by an opening ratio approximately equal to 0.5, and the ratio between vertical stresses of the two models (with/without rigid floor) are equal to 20%, 9%, 11% and 17%, respectively on the isolators 23, 24, 25 and 26. The diagram of Figure 7, in the case of the simple portal model with opening ratio equal to 0.5, provides values of the stress ratio equal to 7.6% and 8%, (respectively for the support b and a) which are quite close to the values obtained for the internal isolators of the macroelement B; for the external isolators the axial force variation is almost approximately double, due to a similar effect arising in the longitudinal direction. It seems that the above considerations have important design implications; in fact, when the artistic feature of the church does not allow the construction of the r.c. floor at the isolation plan, local stiffening interventions and simple ties can be placed at the base of the macroele-ments; however, the effect of the deformability of the macroelement-tie system should be taken into account when the axial load acting on isolators, due to the only vertical load condition, is computed.

5. CONCLUSIONS

In this paper, some design aspects on the base isolation system have been evaluated. A dimensioning procedure and a list of checks on the devices have been tested on two study cases, namely the church of St. Giovanni a Mare and the church of St. Ippolisto. Modal and time history analyses of the base isolated churches have been carried out. They have allowed to define vibration modes, periods and participating masses, shear distribution among the isolators and the church macro-elements, the maximum and minimum axial stresses in the isolators. In particular the attention has been focused to assess the differences between two models, with and without rigid diaphragm connecting the isolators. The presence/absence of the rigid floor does not imply variations in terms of accelerations, displacements and forces in the upper part of the construction, but only different values of the normal stress on the isolators for vertical load, due do the different deformability of the upper-structure. Simplified models for describing this effect have been presented and a parametric analysis has shown that, for church masonry macro-elements characterised by very large opening ratios, the vertical stress on isolators significantly varies considering the presence/absence of rigid floor at isolation level.

As a design implication of the above observations, it can be stated that, when the artistic feature of the church does not allow the construction of the r.c. floor at the isolation plan, local stiffening interventions and simple ties can be placed at the base of the church macroelements, provided that the effect of the deformability of the macroelement-tie system is taken into account.

ACKNOWLEDGEMENTS

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ELASTIC SUPPORT SYSTEMS FOR THE PRESERVATION OF CULTURAL HERITAGE

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1. INTRODUCTION

There is a huge amount of structures worldwide which are classified as national or international cultural heritage. Most of them are quite old and their damage or destruction would mean a tremendous loss of values. Older and younger history has shown that there is a certain threat for these structure caused by the interaction with the supporting subsoil or foundation. Damage may be caused by high vibrations levels, earthquakes or differential settlement of the subsoil. Cracks and plastic deformation in the structure or even overturning and subsequent damages can be observed in many examples of cultural heritage which were not protected.

The following paper deals with elastic support systems which are frequently used to protect structures against the interaction with the subsoil. The structure is elastically or dynamically uncoupled from the static or dynamic behaviour of the foundation. These technologies are usually used for new structures but they can also be applied for existing ones. Here, this task is much more complex as the structure may be very sensitive and is not allowed to experience any damage from the corresponding construction work. Properties of spring elements and dampers are described and details of executed projects are discussed.

2. ELASTIC SUPPORT SYSTEMS

Spring elements with helical steel springs possess linear-elastic behaviour in both horizontal and vertical directions. Therefore, their numerical description is comparatively simple and the behaviour of the structure on these devices can easily be assessed. The elements are carrying the dead load of the structure and are designed to have sufficient safety margins to bear also additional loads from differential settlement and/or earthquakes.

The superstructure has no rigid connection to the surrounding and hence, the behaviour of the structure is mainly determined by properties of the support.

Figure 1 presents two examples of elastic support systems. The left hand side shows the basement of an office building which is entirely supported by spring elements. There is a train passing very closely and without elastic support the people inside would be disturbed by high vibration levels. The right picture shows the installation in a high seismic area. There are many examples using these technologies to protect structures against earthquakes.

Figure 1 – Elastic Support Systems is Basements of Buildings

The design of the support system depends on the corresponding target performance of the structure. Usual vibration protection systems for buildings work in a range of 3 to 4 Hz in the vertical direction. With these values high frequent traffic induced vibrations can be isolated very efficiently. The devices have the same order of stiffness in the horizontal direction to ensure the proper operation also during strong wind excitation and other horizontal forces. In areas without earthquake no dampers are necessary to control the seismic responses.

In seismic areas the strategy is modified to take into account also the horizontal components of earthquakes. The horizontal stiffness values of the spring elements are very low. Hence, the corresponding motion of the structure can be controlled very efficiently and consequently, a very low level of internal stresses can be assured for the structure. The application of usual standards is no more valid for the protection of cultural heritage. Common norms allow a certain amount of damage; life safety is the most important target here. This is not sufficient for the protection of cultural heritage as also slightest damage has to be avoided in any case.

3. VIBRATION ISOLATION STRATEGIES

Basically two strategies of vibration isolation can be distinguished: Active and passive isolation systems. Active isolation is more effective as the devices are directly located at the source of vibration. Any further transmission of vibration is of minor importance. Typical applications can be found for the isolation of machine vibrations and spring-mass systems for trains and trams.

Passive isolation systems are applied for sensitive machinery, equipment and buildings. High-frequency excitation, which may disturb or endanger the structures, are filtered by low frequency support systems. The isolation is frequently arranged below the main part of the structure above the basement. There are more than 100 buildings worldwide nowadays using these technologies. The left-hand side of Figure 2 shows an example for active isolation with a spring-mass system and the right sketch gives a typical arrangement of the spring devices in the basement of a building.

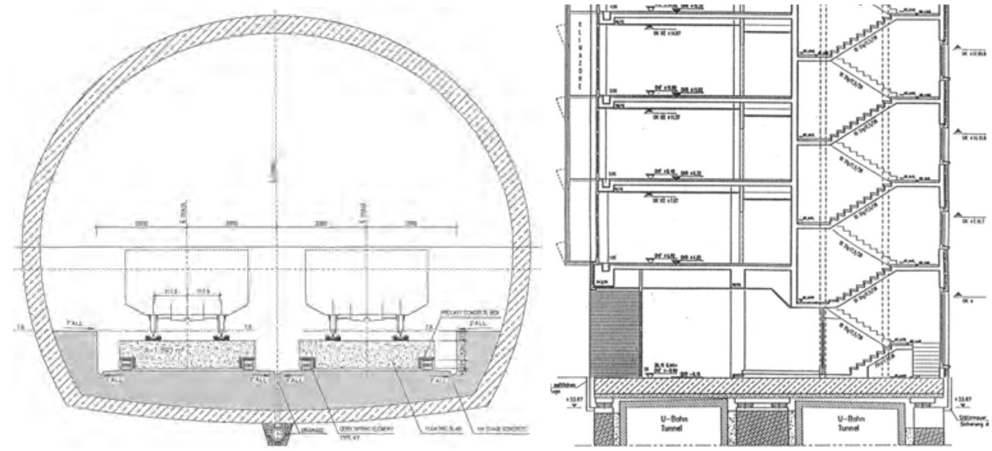


Figure 2 – Floating Trackbed and Typical Isolated Building

Structures of cultural heritage are frequently sensitive in regard to vibration. Certain levels of vibration may cause cracks, e.g. in masonry. As a consequence the structure is not optimally protected against other environmental effects. Thinking about the historical age of these monuments and the location near to well known vibration sources this risk has been underestimated during the recent decades. There are many examples of old buildings like mosques or churches located near to frequently operating tram lines. Measured vibration levels in most cases are much higher than the recommended values according to different standards, e.g. DIN 4150, part 3.

Figure 3, left side, shows the active isolation of a tram switch in Germany. An old church in a distance of about 25m was affected by vibrations caused by trams crossing this switch. Severe crack pattern occurred during the decades and made retrofit measures mandatory. The switch was separated on a foundation block which is supported by a low frequency spring system now.

The right hand side of Figure 3 presents an old building which was retrofitted by helical steel springs. Trains caused heavy vibrations without the isolation system which was installed subsequently.

Figure 3 – Isolation of a Switch near an Old Church and Isolated Building

4. EARTHQUAKE PROTECTION SYSTEMS

Special spring- and damper systems have been developed to protect structures against effects of earthquakes. Many examples can be found worldwide for machinery, equipment, buildings and other structures. Low-tuned, mainly horizontally working systems mitigate effects like exceeding the limit stress values, related crack development, the appearance of plastic zones, or even overturning.

The effects of seismic control are well known and a short description can be found in prEN 15129, 2004 ("Anti-Seismic Devices"). Here, this technology refers to the design of devices that are provided in structures with the aim of modifying their response to seismic action. The modification may be obtained by increasing the fundamental period, modifying the shape of the fundamental mode, increasing the damping and/or limiting the forces transmitted to the structure.

In Figure 4 typical examples for the arrangement of these devices are shown. Left hand side displays a historical statue in a museum, the right side a typical detail in the seismic retrofit of buildings. The basic layout of the systems is similar as the structure to be protected sits on top of the system. The structure is supported by a foundation which provides sufficient rigidity to the structure. The foundation can be designed like a rigid block or a base mat with or without beams. This foundation is elastically uncoupled from the supporting base mat or pit by springs and dampers. According to the physical arrangement these strategies are called Base-Control Systems. The corresponding devices are designed in line with the seismic action and respect the target performance during the prescribed earthquake events. In many cases the prescribed seismic demands exceed the values according to the national or international codes for buildings.

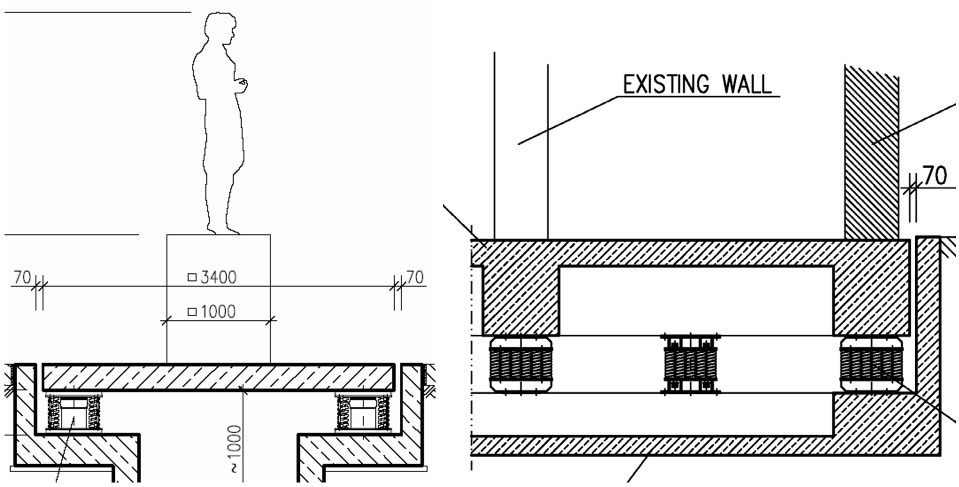
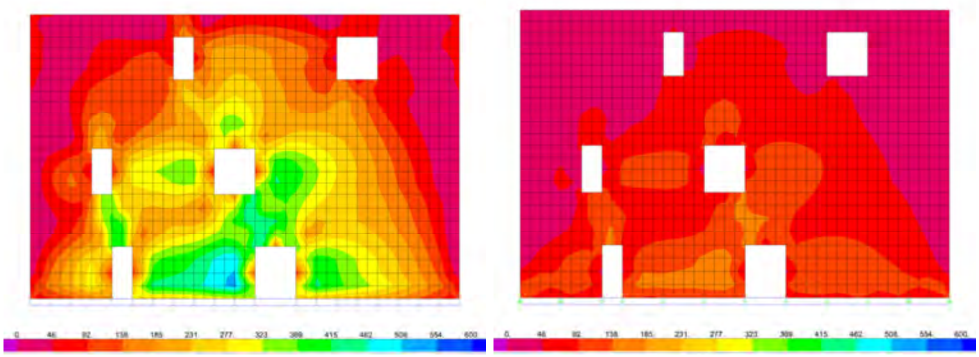


Figure 4 – Base-Control Systems for Statues / Retrofit of Historical Buildings

Figure 5 refers to the seismic efficiency of Base-Control Systems in regard to the stress reduction in a historical masonry wall. The left hand side provides some idea for the stress development of the unprotected wall during a seismic event (envelope). The efficiency becomes obvious by comparing the corresponding stress values with the right hand side. Here, a reduction of more than 50% can be estimated and consequently, the risk of cracks and other damage by earthquake becomes significantly less.



Without BCS With Base-Control System

Figure 5 – Stress Reduction in a Masonry Wall of a Historic Building

5. PROTECTION AGAINST BUILDING SUBSIDENCE

Building subsidence occurs in many regions on earth. The reasons and the expected soil motion may be different from case to case. Mining areas are in special danger but also historical buildings may be concerned by excavation activities, e.g. by drilled tunnels below for the expansion of traffic networks.

Figure 6 shows the St. Remigius Church in Bergheim, Germany. Parts of this building are more than 600 years old and some damage occurred during the last decades. An example of the crack pattern is also given on Figure 6.

Figure 6 – St. Remiguis Church in Bergheim, Germany, Observed Cracks

Detailed investigations of the damage lead to differential subsidence as reason for the cracks. The geological fault line affected the east part of the church (see Figure 7). The idea for the retrofit stretegy was based on the installation of vertically elastic layer. By using springs between two separated foundations systems, the structure becomes more robust against vertical motion as a deviation below the devices causes only a little change of the corresponding spring force. This principle is illustrated in Figure 8. When a certain amount of vertical motion has been recorded, the hight of the spring system can be adjusted by inserting or removing shims. Figure 9 shows some parts of the construction works and the final situation with spring elements.

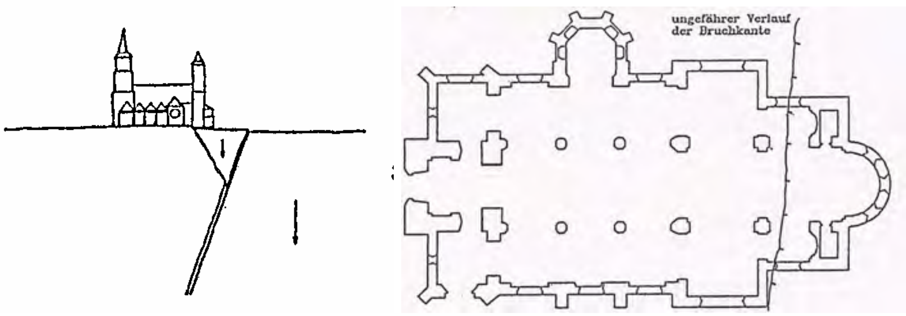


Figure 7 – Geological Fault Line in the Section and in the Plan

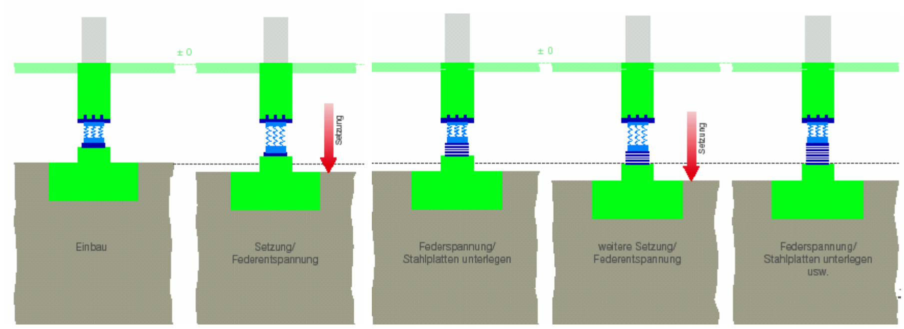


Figure 8 – Principle of Spring Systems against Subsidence

Figure 9 – Structural Upgrade Works and Final Spring Support System

6. CONCLUSION

Elastic support systems may successfully be installed to control the behaviour of structures of cultural heritage. Differential subsoil settlement, vibration problems and seismic danger can be faced in an effective manner. Springs and dampers have been introduced as devices in the single chapters of the present paper. Several examples have been shown and the basic operation of the system has been discussed. The installation of devices offers several advantages and can be characterized as follows:

- The system is very effective in reducing the seismic demands in the structure. There are many applications worldwide which have proven its worth during various earthquake events.
- Springs elements provide low natural frequencies of the entire structure and hence, the vibration isolation efficiency is very high. The installations may follow the active or passive isolation technology.
- Due to the vertical flexibility of the devices the building keeps the original height in all of the constructions phases and after the retrofit.
- Due to the vertical flexibility the forces on the foundations are comparatively small. The vertical load on the bearings is proportional to the vertical relative displacement.

- Spring devices are usually prestressable. Hence, the installation process is easy and there is always the possibility to modify the support system, if necessary. A height adjustment is very important in subsidence areas.
- There are no bolts required for the fixation of the devices at the super-/substructure. Usually self-adhesive pads are sufficient.
- The resultant horizontal motion during the strong earthquake motion phase is in the range of about 25–45 mm which is small when comparing with technologies using rubber bearings for example.
- Usually more devices in order to support the building are arranged. In this manner the distribution of the dead load from the bearing walls to the foundations is very uniform. The usual approach of a pile support or significant strengthening of the subsoil properties may not be necessary.
- The load from the building is distributed very smoothly using a closer distance between the support locations.

Spring and damper devices are always designed for the specific purpose. The elements vary especially in the bearing capacity, horizontal and vertical stiffness properties and damping.

The structural upgrade work for the subsequent installation of a elastic support systems below structures of cultural heritage has to be prepared and carried out very carefully. Any mistake would lead to damage and an irrecoverable loss in the worst case as the foundation system of the structure is manipulated.

7. LITERATURE

DIN 4150-3	Structural Vibration - Part 3: Effects of Vibration on Structures
DIN EN 15129	Anti-Seismic Devices
DIN EN 1998	Eurocode 8: Design of Structure for Earthquake Resistance

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ORIGINAL SOLUTIONS FOR STRUCTURAL AND FUNCTIONAL REHABILITATION OF MASONRY BUILDINGS

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ABSTRACT

The paper presents strengthening solutions and elements of functional rehabilitation for a certain class of 2-4 storied historical buildings of masonry structure. In order to protect the ornamental plastering of the ceiling, an original technology of rehabilitation for large spanned timber slab structures was performed. The worked out technical solution was applied on several buildings. It is presented on the case of a historical building from the 19th century. This case also gives the opportunity to emphasizing the functional improvement of the building.

1. INTRODUCTION

At the time of 17th to 19th centuries, from Baroque to Eclectic era, a large number of 2-4 storied masonry buildings were performed all across Middle-East Europe. Most of them public buildings are functioning even today. So there are the Palace of Justice in Odorheiu-Secuiesc (Figure 1) and the Palace of Justice in Miercurea-Ciuc (Figure 2), both in Transilvania.

Generally, the supporting structure of these buildings consists in: brick or/and stone-walls and pillars as vertical supporting elements, respectively brick or/and stone bridging elements like masonry arches and vaults, barrel vaults with metal beams or timber slabs as horizontal bearing structures.

Many of these buildings present serious damages today, influencing the reliability of them concerning serviceability limit state and even ultimate limit state requirements.

Figure 1 Palace of Justice
in Odorheiu-Secuiesc

Figure 2 Palace of Justice
in Miercurea-Ciuc

In previous studies [1] [2] different types of historical masonry vaults and slab structures were defined and classified. Their characteristic deficiencies and damages were also put into evidence. A convenient and objective approach of the technical state of the building was proposed and solutions of strengthening were presented.

At this time, the purpose is to present some special strengthening techniques corroborated with architectural and functional demands in the frame of the general rehabilitation of the building. The necessity of functional improvement of the old buildings has to be also discussed. These purposes are developed through a significant case (Figure 1).

2. DESCRIPTION OF THE BUILDING

2.1. Historical considerations

The Palace of Justice in Odorheiu-Secuiesc is a representative example of many administrative buildings constructed in Transilvania during the 19th century. These buildings responded to the main process of reorganizing and developing the whole system of justice. In this order, the Palace of Justice in Odorheiu-Secuiesc was finished and put in function in 1835.

2.2. Structural and architectural description

The palace initially housed not only the trials, but also the prison. Consequently, the organization of the interior spaces corresponded to the functional requirements of that time. It is a two-storied building with basement (Figure 1). The structure of the cellar is of masonry stone walls sustaining brick arches and vaults (Figure 3).

Figure 3 Masonry stone walls, brick arch and vault

The structure of the ground floor and first floor consists in brick masonry walls and timber slabs. The roof structure is of main timber trusses of Eclectic type. The whole building can be classified as Eclectic, with late Baroque influence.

3. THE GENERAL REHABILITATION PROCESS

The rehabilitation of the Palace of Justice was carried out in the years 1999 – 2003. The diagnosis of the building put into evidence the necessity of a capital repair and also consolidation of some subassemblies of the structure in order to reassure the resistance and serviceability of the building. On the other hand, actual demands of functionality imposed its extension, by adding a new block to the old building.

3.1. Consolidation of the masonry vault system over the basement

The masonry vault system of the basement is supported by masonry arches and walls (Figure 3). It is to be mentioned the pronounced flatness of the brick arches.

The examination of the masonry vaults and arches put into evidence two main aspects: cracks and breaks with detached bricks on the one hand and important vertical displacements on the other hand. Obviously, the vault system had to be discharged. For this purpose, a new independent slab system was performed over the initial bearing system (Figure 4). It consists in a reinforced concrete slab on metal beams leaning on the stone walls. So, the vaults and arches have to carry only their own weight. The detached and destroyed pieces of brick were replaced with new ones and the masonry joints were filled with lime mortar with powdered brick as hydraulic additive.

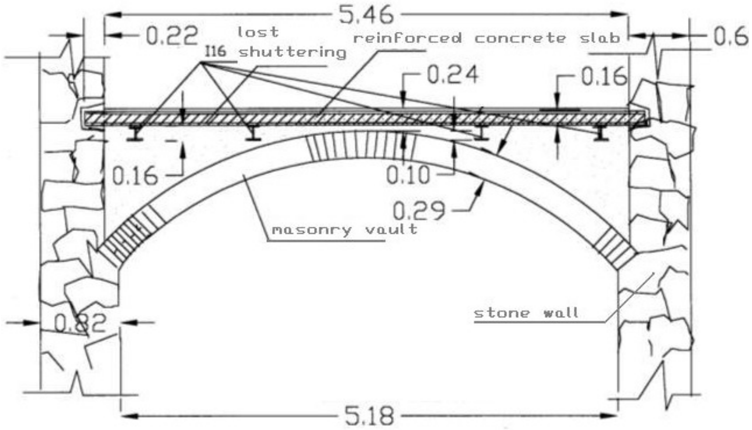

Figure 4 Consolidation of the slab system over the basement

3.2. Consolidation of the timber slab system over the first floor

The ordinary timber slab over the ground floor and first floor consists in wooden beams bridging the shorter opening between the supporting masonry walls. Their diagnostic, repair and consolidation represented usual engineering tasks. The strengthening solution was: discovering the timber beams, replacing or repairing them, their treatment with antiseptic and fire protection substances, using them as shuttering for the new reinforced concrete slab. The ventilation of the old timber slab was assured by providing a slit between the lower face of the old timber slab and the false ceiling attached to it. Holes of ventilation were provided along the perimeter and around the lamps.

The main technical problem was the rehabilitation of the slab system over the Great Judgement Hall on the first floor. Originally, it is a timber slab system covering a surface of 11.50 x 10.50 m^2. The slab system consists in a girder network of main and secondary timber beams supported by the perimeter walls. A number of three main beams crosses a span of 10.50 m, placed at a distance of approximately 2.80 m one from the other, thus creating a number of four 2,80 m long openings for the secondary beams. Double boarding is fixed on the upper, respectively on the bottom face of the grid. The original ceiling was realized by a decorative plastering on a reed sheet fixed to the inferior boarding. On its upper side the slab structure was completed with thermal slag insulation and brick covering.

The most inconvenient phenomenon was the great deflection of the slab, respectively of the main beams. The measured vertical displacements at the middle of the beams were between 11 and 14 cm. This phenomenon caused serious damages, i. e. visible cracks in the ornamental stucco plastering of the ceiling (Figure 5).

Figure 5 Plastering in stucco of the ceiling

The aim was to find a technique of intervention ti diminish the deflections saving the ornamental plastering of the ceiling too. For this purpose it was necessary to bring the timber slab back near to its initial form. In this respect, at first, the slab has been discharged taking away the vertical loading given by the floor (thermal insulation and bricks). Secondly, the middle of the main beams had to be lifted for diminishing the vertical deflections. The solution was to build over each main timber beam an independent reinforced concrete girder (Figure 6) supported by the marginal walls.

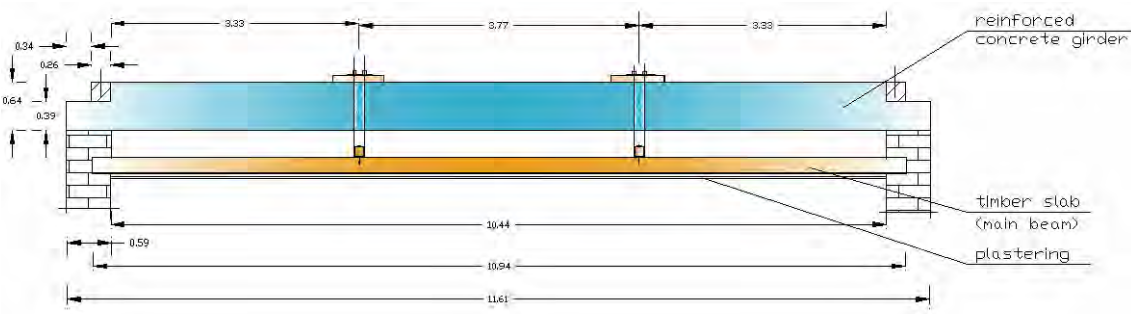

Figure 6 Lifting girder

An additional temporary sustaining system was conceived to sustain the shuttering, the reinforcement and the cast concrete of the lifting girder. It consists in a triangular timber structure with a tie-rod (Figure 7 and Figure 8) to which the vertical loading is transferred by a series of vertical metal bars.

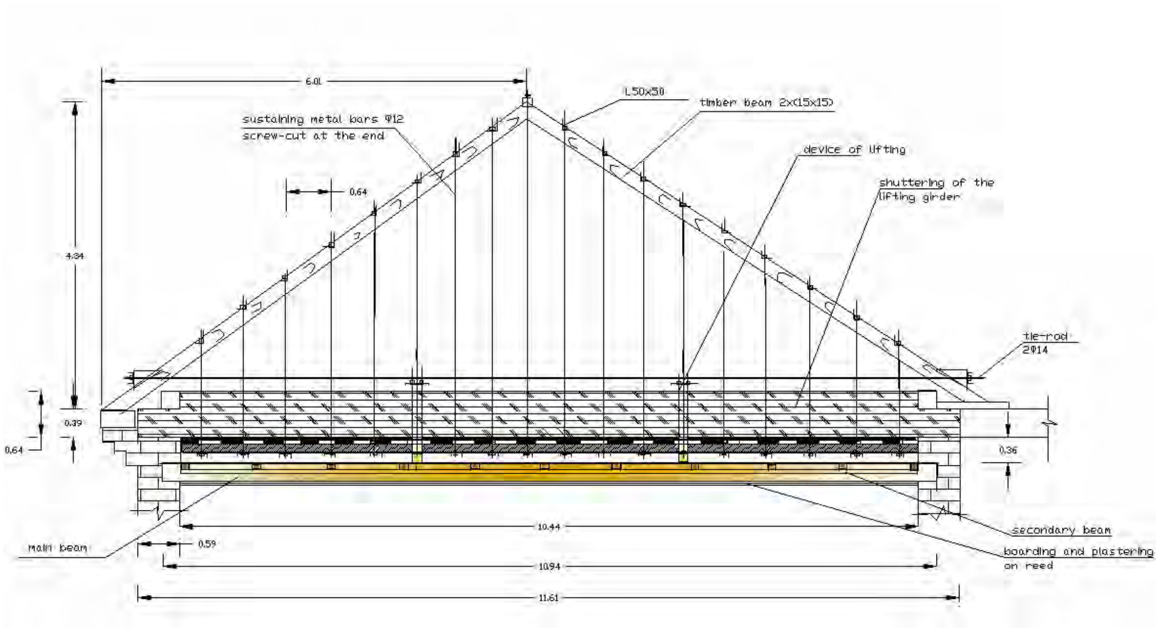

Figure 7 Additional supporting structure of the lifting girder - design

Figure 8 Additional supporting structure of the lifting girder – photo

A transverse section of the whole lifting system, after striking the reinforced concrete girder, is presented in Figure 9.

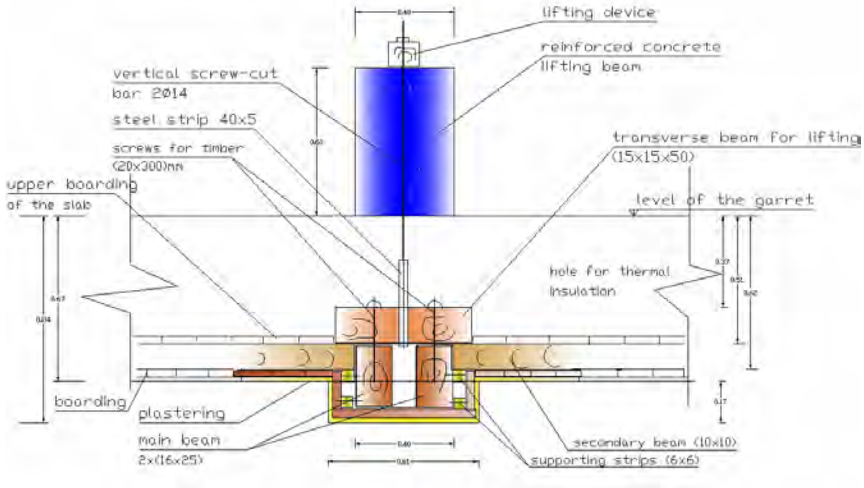

Figure 9 Transverse section of the lifting system

The operation of lifting can be performed in two ways:

(a) Lifting the slab before casting the girder, using the additional timber structure for lifting; in this way the bottoming of the two lifting bars is made automatically by the concrete hardening; a relatively short time for the operation can be assured (7-8 days);

(b) Lifting after the execution of the reinforced concrete girder; the lifting bars pass through the girder by former embedded tubes; it has the advantage of a perfectly controlled way of lifting and bottoming the lifting bars on the girder surface by screwed devices; the time of operation is relatively long because of the hardening time of the concrete;

In this way the middle of the main beams could be lifted with 8 – 9 cm without displacements of the supports. This rising of the axes of the beams was enough to close the cracks of the ornamental plastering and finally permitted a proper restoration of the ceiling.

3.3. Functional rehabilitation

For improving the circulation flux in the building and also responding to the actual requirements concerning emergency evacuation, a new staircase was added to the old building. The originality of the solution consists in the fact that the architect didn't try to take again the initial style of the building. On contrary, aiming to a maximum functional and structural efficiency, a very modern solution in reinforced concrete and glass was adopted (Figure 10, Figure 11).

Figure 10 New staircase - interior view   Figure 11 New staircase - exterior view

4. CONCLUSIONS

The study put into evidence one of the major dilemma of the rehabilitation process: the necessary balance between preserving the valuable historical characteristics of an old building and today's requirements concerning the safety, serviceability and functionality of the building.

The paper presents an original technical solution for consolidating large spanned timber slab structures saving architectural values as well. The solution consists in diminishing the vertical deflection of the slab by using a special lifting system.

Modern architectural solutions can be successfully used in order to improve the functionality of the historical buildings, without disturbing, on the contrary, emphasizing their historical character.

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GABRIELLO CHIABRERA'S OPERA HOUSE IN SAVONA AND ITS ACOUSTIC VAULT

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ABSTRACT

Functional recovery intervention, static consolidation and restoration of the acoustic vault in Gabriello Chiabrera lyric theatre, built in Savona, Italy, planed by Carlo Falconieri in 1849, had had for object the acoustic vault or "velarium" which, for its constructive characteristics, belongs to the light typology vaults, realized with a mat of canes nailing on wooden centerings. The wooden centerings give form and resistance at the constructive system. Constructive technique of light vault was just explained by Vitruvio and it consisted in a uniform finishing gotten with many coats of mortar. While structural centerings are hanged from the girders of the over floor or roof by little boards or wooden roundish supports. This system allowed to create a "camera" or a space between the over floor and the below vault. The dramatic event happened in October, 1999 ended with the detachment of 40 square meters of plaster and mat of canes from wooden structure hanged up stalls. It had compromised the structural characteristics of ceiling, the acoustic of the hall, the image of the Neoclassical theatre as the architect Carlo Falconieri's original idea. The decision of intervening and replacing the part of collapsed vault where it vas and giving a new continuity and solidity to the acoustic vault had required many difficult whether about techniques or procedure of intervention or conceptual. Among these, first of all the necessity to have more and more information about techniques which were old and unknown because of it was realised in 1849, then the necessity to know why it was collapsed. At the end the complexity of necessary procedures for reassembling the vault, repairing the structural unit at 20 meters up on the stalls. This operation had been a challenge accepted by architects and workers with the target of recovering the solidity and the functionality of the theatre.

1. THE PROJECT OF THE OPERA HOUSE

Gabriello Chiabrera's opera house is an interesting neoclassical building realized by the architect Carlo Falconieri from Messina for Savona's Communal administration. Town of Savona is a seaport of the North-West of Italy.

Falconieri was entrusted with the task of building theatre owing to the victory in 1849 after competitive examination announced by «Regno di Sardegna». It was a competition that included architects of other Italian States [8. Ragazzi, F. 1991, pp. 64-77]. The plan of theatre provided three rows of boxes and a gallery for the comprehensive cost of one-hundred-ten-thousand "lire".

Falconieri's plan was modified during the works by the architect Giuseppe Cortese, who was also the work director. The project consisted in an ample front which was 27 metres long and 57 metres deep, the maxim length consented by the Communal building site [1. Aiolfi, R. 1963, p. 40] (Figure:1).

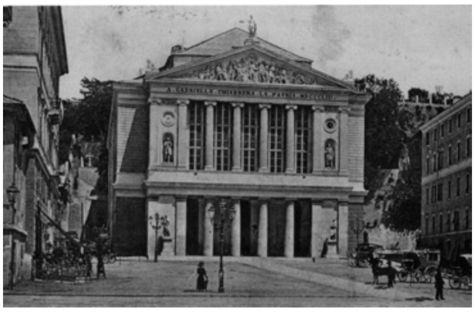

Figure 1. Front theatre in an ancient image

Plan of the opera house had the shape of horseshoe which was a typical form of lyric theatre and it had 380 seats in the stall, 370 seats in the balcony consisted of three rows of boxes laid one upon the other and 250 seats in the gallery. The total capacity was of 1000 seats [11. Tondi , L. 2005, p. 32].

Neoclassical typology of building, which was a typical style used for other Italian opera houses, needed the concert hall was concluded by a *velarium* or an acoustic vault. It was realized for consenting a harmonious diffusion of the sound. It had a great lightness and an a-symmetric bent shape which avoided the eco or reverberation. Intervention would analyse the complex problems which had characterized the built of *velarium*, the changes in original balance with time, the reassembling and repairs.

2. CONSTRUCTIVE CHARACTERISTICS OF ACOUSTIC VAULT

After the ruinous collapse of the global vault happened 11th October, 1999 that strewed about 40 square meters of *velarium* , 1/3 of the all structure - in the stall and in the boxes, we decided to attend the structural unit and to recover the complex constructive system by replacing *in situ*, 2000 fragments which were fallen down [10. Scunza , R. 2005, p. 13].

By studying different sources, the bibliography that was found about, the iconographic and archives we had finally found the acoustic vault was realized by a genius system, showed in Vitruvio's *De Architettura*, with the definition of «Greek canes – camera» [12. Vitruvio Pollione, M.1567, p. 260] .

The elements constitute the canes-camera are simplex and they are connected in a genius way, they allow to cover considerable spans among each supports once they are rightly assembled. Work method consisted in riveting Greek canes under wood centering forms which fastened and gave the characteristic curved shape (Figure: 2) [7. Galliani, G.V., voce «incannicciato», p. 419].

Figure 2. The light mat canes structure nailed an wooden matched and curved centerings which form the characteristic shape of the vault

The constructive system, characterised by lightness considerable resistance, was stiffened further on by wooden elements, named «distanziali» which means spacers. They are nailed between centering and centering in a transversal position; they develop the delicate task to control equidistance between centerings and to limit torsion and distortion of all system. The light structure is hanged up by little wooden boards or wooden roundish supports to the roof or attic overhanging girders.

Warp and weft of canes are put in an unequal way, the first are thicker and cadenced, the second are thinner and sparse. They consented to compose a mat resistance in interlacement but with expansible meshes, that are nailed on wooden centerings. To intrados of the light vault, gained in such way over described, it is finished with many coats of mortar lime plaster and it can be about 0,04 metres thick.

This system could carry out a sound box and defined the materials needed for building such great light under roof.

It was brought to perfection by an uniform spread of fine plaster containing mortar of lime that fastened the Greek canes each others and gave evenness to the sound box.

3. REASONS OF THE COLLAPSE

The careful analysis of the acoustic vault structure, of the parts remained hanging from and of the fragments of plaster and canes, consented to understand some observations about constructive system and about deformations that can be the reason of the detachment of the ceiling from the wooden centerings and the collapse.

Canes-camera structure was built beginning from perimeter of wooden boards shaped in horseshoe, by the following assembling stages:

n°1, about the pair of thin boards (9,37 m length; 0,23 m height; 0,10 m wide) put starter from the entrance towards the stage , hanged up the attic;

n°2, given prominence to transversal stiffening made by putting two short boards (2,36 m length; 0,23 m height; 0,10 m wide), fitted in the first two drawing a square which is 2 m by side;

n°3, built a sturdy wooden drum (0,27 m height of circular crown; 0,23 m wide; 1,00 m inside radius) which is inscribed inside the square as further stiffening of the structure;

n°4, allowed to complete the base structure with two couples of curved centerings (1,90 m and 2,48 m length; 0,23 m height; 0,10 m wide) which linked the longitudinal boards, the transversal once, and the drum with the horseshoe perimeter;

n°5, put in radial arrangement 80 centerings of different length and radius in order to follow the horseshoe perimeter, on horizontal plane and to follow the shell bent shape of the vault, on vertical plane (Figure: 3).

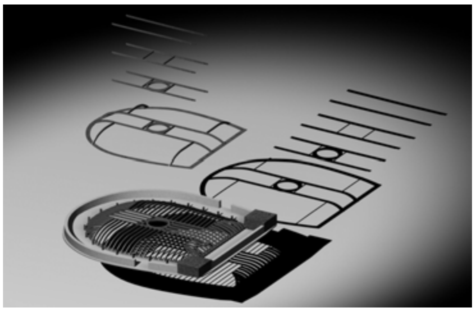

Figure 3. Three D model with graphic simulation of assembling stages

A first look brings out the absolute absence, in the wooden structure of the vault, of the elements named «distanziali» (spacers), normally used which connected the centerings each others for preventing centering axial deformations.

A second look is about deformations on horizontal plane of the centerings complex constructive system.(5,50 m maxim length) with swinging values between 30 mm and 60 mm. This problematic has involved the next detachment of nailing canes mat which had a swinging detachment from 22 mm to 37 mm until to 45 mm, in few areas next to the gap created on the vault next following.

Since canes mat is putting on wooden centerings thank to use nails, which are 35 mm length and hammered in the plaster for about 15 mm deep, it is easily to understand the attachment of the mat to the wooden centerings failed in some areas of the ceiling and the mat was «in bando» over the stalls.

Traction efforts support to the matter were very strong and when the connection between mat canes warp and weft canes mat and the mortar plaster resistance failed too, a part of the ceiling fell down.

A third consideration was found in information taken after structural survey. It allows to considerate carefully the nailing that even if frequent (250 mm distance between each others) and fixed with alternate pitches, did not contrast at all the efforts transmitted by torsion which are impressed in wooden fibres of centerings. They represented the weak point of the system , in some cases they are blown up to the support (Figure: 4).

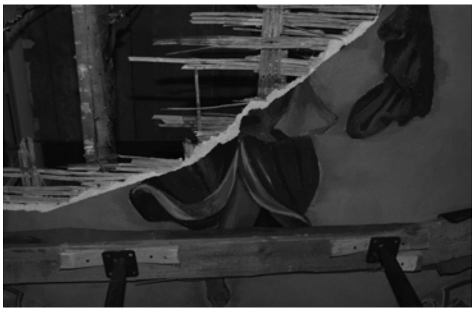

Figure 4. Detail of big gap edges

4. STATIC INTERVENTION OF CONSOLIDATION AND RESTORATION

Reassembling operations of fragments fallen down in the collapse are developed in the following order:
fragment fallen down reassembling with the help of a photograph of the *velarium*, when was yet complete, and reproduced full scale on PVC support, laid on stage, transformed in yard space (Figure:5);

Figure 5. See of the «graticcia» of yard on the stage, during reassembling stages of recovered fragments (photo by Vassallo)

the operation makes to the back of fragment in order to get rid of impressions left by canes mat warp and weft on mortar attachment, consents the perfect join among edges, sometimes unequal or incomplete;
precautionary intervention of anti-parasite and of wooden structure surfaces cleaning in order to preserve and consolidate wooden fibres of centerings;
consolidation of part of ceiling which was staying already on after collapse in
11th October, 1999, executed with the logical of recovery, taken aim at reinforcing and at setting the existing structure in safety without replacing consolidation consisted in integrating resistance characteristics of elasticity, keeping steam permeability and keeping reply at solicitations made by sounds on plaster surface;
arrangement in longitudinal and transversal direction, as regards to the all, of tissue bands made by fibreglass unidirectional stuck with epoxy resin;
arrangement of «distanziali» (spacers) for limiting deformations and tensions to each centerings
laying of fragments involved to the ceiling collapse with the *auxilium* of Plexiglas cons-shape, curved for doing curved shape to the vault, in order to satisfy information deduced to survey operations (Figure:6);

Figure 6. Fragments lay down lean on Plexiglas panels (photo Vassallo)

widespread consolidation and punctual anchorage of fragments and of detachment parts, brought to perfection by using 150 connectors, of different sections stainless steel and length in order to satisfy different dimensions and position in the space of fastened fragments (Figure: 7).

Figure 7. Vault fragments reassembling on cons-shape realized with insertion of stainless steel connectors

4. CONCLUSIONS

Replace vault fragments in the original position and integrate themselves and consolidate the structure have been as building an open bridge to the past and as travelling over stages and moments of the 1849 yard, discovering constructive experiences of «conventional technology» which are definitely unused and lost in times [4. Manzini, E. 1985, p. 6].

Structural study avails itself of a careful survey with tolerances round millimetres, obtained with laser equipment, integrated with a method of analysis defined, by some opinion schools, «systemic analysis». Method committed the comprehension of old building – as an architectonical organism [9. Rava, G.P. 2005, p. 133] which had its own complexity and its various relationships existed among structures, interior spaces and functions – to a reasoned selection of elements that compose the organism and to their order recognition according to material sequence, to constructive system, to resistance system, space and functional system [6. Galliani, G.V., De Battè, B. 1990, p. 7].

Besides method defined, with words «structural conception», the mean idea according to original structure has been articulated in its resistance parts inside architectonical organism [5. Galliani, G.V. 1987, p. 779]; words «logical structure» mean whole factors, also critic, which are necessary to recognise and to interpret transformation succession during time which have involved the structure [2. Buti, A. 1999, p. 445]. Continuous comparison with factors of systematic analysis, integrated with study of structural and logical conception confers to words «way of knowledge about building - art» new meanings and different relation boundaries [3. Di Pasquale, S. 1996, p. 59]. Method constitutes in a precious inventory that could be used for understanding organism and complex transformations, which had changed balances until the collapse. Method also drives recovery operations with employment of new materials and actual constructive techniques in direction to the «greatest preservation» in transformation in order to getting a new use in continuous with building history (Figure: 8).

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Figure 8. Vault restored

ACKNOWLEDGEMENT

A team of architects, workers and researchers have led the present work. They worked narrow and compared different ideas, proposes and different operative ways so they could have realized the intervention. First of all, must be mentioned work of Supervising architectonical properties and Liguria's landscape, carries on by Regional Director, Doctor Liliana Pittarello and, in then, by Supervising Architect Maurizio Galletti who was succeeded by Architect Giorgio Rossini during the delivery stage of works; both availed archives researches led Doctor Luce Tondi and by the work created and led on yard by Architect Rossella Scunza, by restorers Paola Parodi and Stefano Vassallo, by Architect Caterina Gardella and by Land-Surveyor Marco di Domenico. Must be mentioned technical and operative ability showed by Enterprises Ger.So S.r.l. and Clessidra S.n.c. (Temporary Association Enterprises) under technical direction of Luigi Soligo. At the end, must be mentioned workers group formed by Doctor Teresa Fior and by Francesco and Mirko Caldini, Architects, who had collaborated, with the author of paper, on execution of survey, on graphic and on «technological conventional» project editing, about Knowledge of materials, old constructive systems and structural logical conception of the vault.

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DESIGN OF STRUCTURAL INTERVENTIONS TO THE BYZANTINE CHURCH OF PANAGHIA KRINA IN CHIOS ISLAND

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ABSTRACT

The Byzantine church of Panaghia Krina in Chios Island has been severely damaged by serious earthquakes during the centuries. Extended rebuildings and conservation works have taken place in the 19th and 20th century. Nevertheless, the church still remained severely damaged demanding a more detailed design project of restoration. This paper presents a brief description of the church and the pathology of its vulnerable load bearing structure, and then discusses the procedure that led to the final design project for its repair and strengthening, based on the results of investigations realized before and during the works, and on the need to combine earthquake resistance and preservation of its authenticity.

1. INTRODUCTION

The church of Panaghia Krina in Chios Island (Figure: 1) is one of the most important Byzantine monuments of the island, dated to the second half of the 12th century.

It is a very well known monument, especially due to the high aesthetic value of the ceramic tile decoration on its facades, directly influenced by the architectural style of Constantinople, and of its frescoes. The church has suffered serious damage during the centuries due to severe earthquakes. Among them, the major earthquake that hit the island in 1881 led to the total collapse of all the vaults and the dome of the narthex, and of the cupola of the central dome in the main church, along with partial collapses and heavy cracking of other structural elements. Important restoration works took place in the end of 19th and the beginning of 20th century, including mainly the rebuilding of collapsed parts of the monument and the installation of iron ties.


Figure 1: Northeast view of Panagia Krina

In the 1970's, after the removal of the outer layer of post Byzantine frescoes (exhibited today in the museum after their necessary conservation) appeared the initial 13^{th} century Byzantine frescoes [1], revealing a more extended network of cracks on the walls. Some urgent local deep re-pointing was then undertaken mainly on the cracks of the external facades, together with in situ conservation of the frescoes, in order to ensure their attachment to the walls, and avoid their further deterioration. The materials and methods of conservation used in the 1970's, do not allow today an extended removal of frescoes and their subsequent reinstatement after the necessary masonry repairs.

The first architectural survey of the monument was undertaken in 1998 [2]. Later, this survey served as a basis for further architectural and structural studies [3, 4, 5]. During these studies it was noted that the church bears an extensive internal timber reinforcement system. These reinforcements were in most locations deteriorated, thus provoking further damage to the vulnerable masonry.

2. BRIEF DESCRIPTION OF THE STRUCTURE

The church belongs to the local single-spaced octagonal type, and is a small-scale replica of the well known Nea Moni Katholikon in Chios Island, (a UNESCO listed monument). The term 'octagonal' refers to the system supporting the central dome, which rests on eight pilasters along the four walls of the roughly square main church. As can be seen in the plan of the church (Figure: 2) the formation of this octagon is not symmetrical. The pilasters are bridged by a barrel vault on the east wing, apses on the three remaining wings, and squinches in the four corners. In fact, the dimensions of the octagon sides are irregular, as the barrel vault's span is larger than that of the apses, which in turn are much larger than the squinches. Towards the North, West and South sides of the church, the loads of the dome and the vaulting are distributed directly to the three perimetric walls while towards the East, to the two freestanding orthogonal pessaries that constitute the front of the Sanctuary. Moreover, the drum of the dome is actually smaller than the vaulting that supports it, and the overhang between them is bridged by a spherical ring.

The twelve windows of the dome's drum are randomly distributed along its perimeter without corresponding to the main axis of the church.

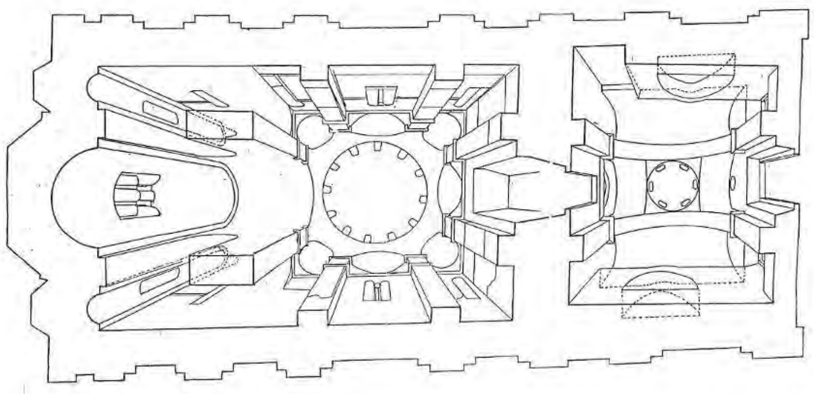

Figure 2: Interior perspective view of the monument

To the West, the narthex is a separate three-sided structure with walls leaning against, but not integrally connected with the walls of the main church. The narthex's vaulting consists of three barrel vaults and a dome. Two wide, but slender arches bear the loads of the dome and bridge the west wall of the church with the west wall of the narthex. There existed also an exonarthex with a timber roof, now collapsed. The above extensive description was considered necessary for the reader to fully comprehend the precarious, asymmetrical and irregular distribution of stresses within the building's load bearing structure.

3. PRESENTATION OF THE MAIN DAMAGES OF THE CHURCH

The pathology of the monument's structure, that was dramatically exposed after the removal of the exterior frescoes' layer (Figure 3), is summarized below:

An extended network of mostly vertical cracks and large areas of disorganized masonry has developed on the spherical ring between the main temple apses and the base of the dome. In many cases, these cracks extend vertically along the walls almost all the way to the ground. Also the eastern vault and western apse have cracked and deformed to such an extent that heavy buttressing had to be applied to their whole surface. A number of vertical large through-cracks in the east wall prove the dismantling of the NE corner of the monument, the upper part of which leans significantly outwards. The front parts of the tall, freestanding orthogonal pessaries supporting the eastern vault, that were partially rebuilt after their collapse in 1881, are again cracked and disconnected from the main mass of the initial pessaries. All the upper parts of the perimetric walls have declined outwards due to the action of horizontal seismic forces that tend to tear apart the four walls of the church.


Figure 3: View of the West wall of the main church

One of the main reasons that provoked the above pathology was the fact that the structural system of the monument presented very serious weaknesses and particularities in critical areas. Those, along with the influence of the earthquakes, the rising humidity and the ageing of materials (decayed timber reinforcements incorporated in the walls, washed out surface mortars, deteriorated decoration tiles etc) resulted in the existing pathology.

4. INITIAL STRUCTURAL RESTORATION SCHEME

Restoration works started in late 2004, based on an initial design project that had comprised all the available data, i.e. the architectural and historical survey of the monument, the survey of its -up to then- visible pathology, three local excavations around the walls in order to estimate the condition of the foundations, a geological-geotechnical survey of the subsoil [6] and the analysis of the characteristics of the construction materials of the monument [7].

All this data was used for the development of a surface finite element model. Spectral analysis was carried out examining the main church and the narthex separately, and then the main church and the narthex connected (Figure: 4).

This analysis permitted the reproduction of the main pathology of the structure and highlighted the necessity for finding a solution to bind the two parts together. When this design project was examined by the Central Archaelogical Council it was decided to urgently start the restoration and during the works to further investigate the locations of the timber reinforcements, in order to examine if there was any possibility to link the two parts of the monument without applying heavy interventions to the external decorated facades.

It was expected that several issues should have to be further explored with the beginning of the works, after the scaffolding had been installed, and the roof tiles removed. Only then it would be possible to complete a thorough documentation of the exact condition of all elements of the masonry (mortars, timber elements, rusted metal ties, infill material, etc.). The need to trace the whole network of timber reinforcements within the masonry, evaluate their condition and try to find how to reinstate their function was deeply recognized. It was also necessary to investigate in situ the original formation of the extrados that were hidden under later infill and find the most adequate and applicable method to connect the main church and the narthex walls.

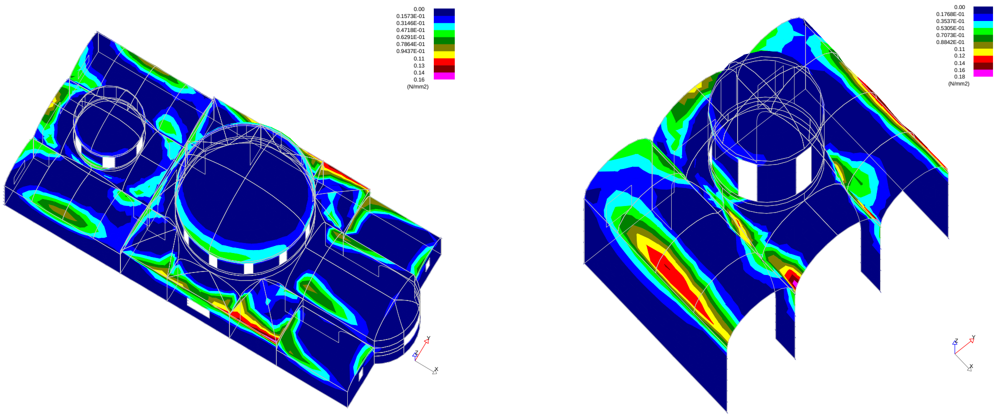

Figure 4: Tensile stress isocurves in the surface FEM model for the connected structure, and the separate narthex

5. RECENT INVESTIGATIONS AND FINDINGS

Two research projects were undertaken by the Directorate for Technical Research on Restoration in cooperation with the National Technical University of Athens. The aim of the first investigation was to determine the extent and precise position of the timber reinforcement network. Equally important was the aim of the second research project, to supplement the already existing surface finite element model and spectral analysis, and take into account the full complexity of the monument’s geometry, the modeling of subsoil and foundations, as well as the particular seismicity of the monument’s area and topography.

The first research project led to the discovery that an extensive network of timber reinforcements had indeed been incorporated in the masonry on four different levels and possibly a fifth one as well. For a detailed description of those see separate paper submitted to this conference by the National Technical University of Athens (Palieraki, Vintzileou et al). Most timber elements were discovered to be either totally missing, or decomposed, and hence incapable of providing the reinforcement that they were initially designed to offer. Another important finding was the particularly haphazard manner of construction of the interior of this three-leaf masonry, in contrast to the meticulously built external facades; a problem further exaggerated by the additional voids left in the place of the decomposed timber elements.

The second research project involved in situ measurements of the monument’s dynamic characteristics, and the time history analysis of a new complex volumetric finite element model that included the foundations and part of the subsoil, and confirmed the results of the earlier analysis, as the areas of maximum stress coincided almost perfectly with the heavily damaged areas of the monument. For a detailed description of the results of this research project see separate paper submitted to this conference by the National Technical University of Athens (Psycharis et al). Further investigations during the restorations works, after the removal of infill over the extrados of the main church, showed that parts of the barrel vault of the east wing and its supports had been largely rebuilt in the past (probably after the 1881 earthquake) as well as the fact that certain surface cracks extended throughout the width of the masonry. An more detailed crack and deformations survey provided further evidence of large disorganized areas of masonry (Figure: 3), as well as heavy lateral displacement and inclination of several supporting structures, sometimes to the extent of 18cm in approximately 9 meters of height. The above new findings confirmed once again that the monument's initial construction alone could account for the damage it incurred in the course of its existence, i.e. that the initial structural design was particularly daring and vulnerable. Moreover, the loss of almost all horizontal timber reinforcements exacerbated the existing weaknesses.

6. FINAL PROPOSED INTERVENTIONS

The results of the additional survey not only verified our initial estimations but also suggested the need for even more extensive strengthening interventions. The restorers dilemma in this case is to carefully evaluate the opposing criteria of safety and preservation, and to decide between either radical strengthening measures that would safeguard the monument against serious future damage (but with significant detriment to its architectural and aesthetic qualities), or milder interventions aiming at a partial reinstatement of the initial load bearing capacity of the structure, thus risking possible (but not devastating) future damages.

The final structural interventions were redesigned on the basis of all this complementary data, and were the result of a multidisciplinary approach and compromise between architects, civil engineers and conservators of the frescoes. Given the uniqueness and aesthetic value of the monument, the final design project opted for a number of modest restoration interventions that would minimally alter the monument's architectural and aesthetic values. However, two necessary measures should be also considered in that case: first, the restriction of public visiting to the interior of the monument, and second the continuous structural monitoring of the monument, both during and after the restoration works.

The main objectives of the final proposals were:

a. To restore the load bearing capacity of all vertical masonry elements, without attempting to rectify the deformed areas. For this it was deemed necessary:

- to implement thorough grouting of all masonry, instead of local grouting as was initially proposed. This intervention requires close collaboration with the conservators of the mural paintings. Hydraulic lime, high penetrability grouts were designed for this purpose.

- to apply systematic stone-stitching along major cracks and in areas of disorganized masonry, even if that demands the temporary removal of small fragments of the existing murals adjacent to the walls’ discontinuities.
- to replace surface timber ties (in the sanctuary walls) with new timber beams.
- to attempt to substitute the role of now missing timber reinforcements, and to connect the longitudinal walls of the main church and narthex, by mounting long metal tie bars in their place (Figure 5). This requires costly and technically challenging techniques of long drilling through and parallel to the walls of the monument. This technique requires thorough grouting, both in order to allow for such drilling to be executed safely, and to ascertain the effective cohesion and cooperation of the tie with the masonry.
- to reseal several of the numerous windows along the walls, and to construct stiff Byzantine-type diaphragms in the place of the existing steel framed windows with glass panes.

b. To connect the opposite walls and the vaults, by applying a network of inox and wooden ties. This includes both the replacement of existing iron ties that had been installed as reinforcements after past earthquakes, and the insertion of additional ties. Yet, some ties in the narthex will remain in place, as they were found to be in good condition, and under stress.
c. To intervene at the level of the foundations by building an external foundation reinforcement, and driving rainwater away from the walls.
d. To reinstate the basis of the drum of the central dome by progressively replacing the remains of the perimetric timber reinforcement in the core of the masonry with natural hydraulic lime mortar filling reinforced with stainless steel bars. Also, to reinforce the area of the perimetric spherical ring under the drum with a system of external bracing made of 4 post-tensioned inox ties.
e. To brace the pessaries under the eastern arch with permanent metal frames, and connect them with the east wall of the church using metal ties (Figure: 6)
f. To reinforce the eastern arch by implementing interconnected fiber reinforced layers of natural hydraulic lime mortar on the extrados and intrados, and adding transverse metal ties.
g. In the case of the two slender arches of the narthex, it was proposed either to construct permanent metal supports underneath them, or to pass tie rods through the main church’s walls and anchor them on the surviving murals. Also, to reinforce the hexagonal base of the narthex dome drum, using a post-tensioned stainless steel ring around its perimeter.
h. All the above measures were combined with a delicate repair of the outer surfaces of the walls, replacing decayed bricks and adding new sealing mortars and bricks only where the original were missing or deteriorated. Careful research led to the design of appropriate mortars for each area, and the selection of new hand-made bricks.

The restoration works are now already in full progress.

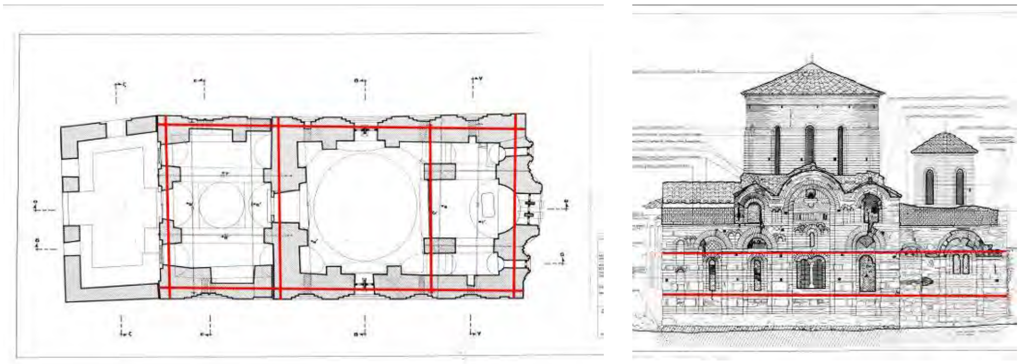

Figure 5

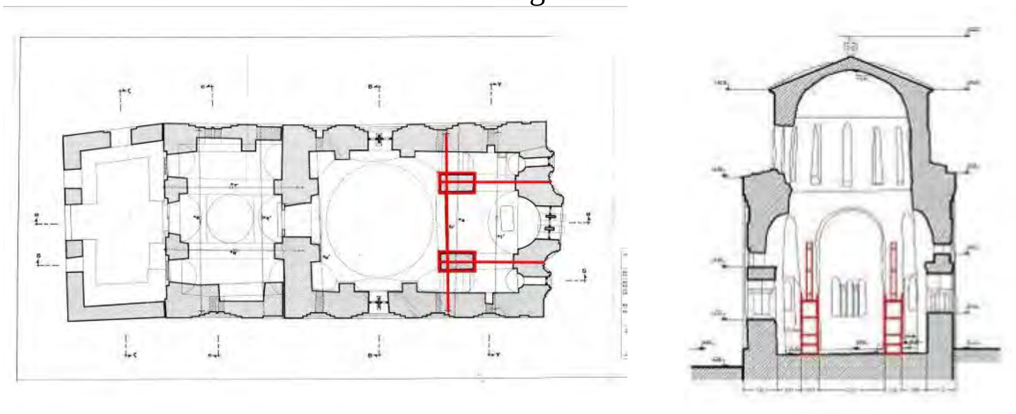

Figure 6

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RELOCATION OF THE HISTORIC CAPE HATTERAS LIGHTHOUSE

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ABSTRACT

This paper discusses preservation efforts at the Cape Hatteras Lighthouse. The Cape Hatteras Lighthouse was built in 1869-1870. The brick beacon is the tallest structure of its kind in the United States and is the primary cultural resource of the Cape Hatteras National Seashore off the coast of North Carolina. The lighthouse received significant media attention during its relocation approximately 1 km (0.6 mi) inland.

The paper discusses the moving of the lighthouse to protect the structure from the eroding shoreline. Conceptual design of the move and development of the approach, geotechnical engineering aspects of the move route, historical materials preservation approaches, underpinning of the existing foundation, the rail system for moving the lighthouse, instrumentation to monitor the stresses in the brick masonry during the move and design and construction of the new foundation are discussed.

1. INTRODUCTION

This paper discusses preservation efforts at the Cape Hatteras Lighthouse. The Cape Hatteras Lighthouse was built in 1869-1870. The lighthouse received significant media attention during its relocation approximately 1 km (0.6 mi) inland. The paper discusses the moving of the lighthouse to protect the structure from the eroding shoreline.

2. MOVING THE LIGHTHOUSE OUT OF HARMS WAY

2.1. Background

A lighthouse was first established on Hatteras Island in 1803. After severe shoreline erosion and deterioration of the structure, a new lighthouse was constructed in 1869-1870. One goal of the new construction was to place the lighthouse "so far removed from the shore as to render it safe from encroachment by the sea." The location selected was 0.5 km (1600 feet) from the shoreline position in 1870. Continuing shoreline erosion had moved the shoreline to within about 40m (120 feet) of the lighthouse by 1980. Historical shoreline regression suggested the lighthouse location would be under water within 20 to 30 years. (Figure 1)

A National Historic Site, the Cape Hatteras Lightstation is managed by the United States National Park Service (NPS). Beginning in the early 1980's, the NPS conducted numerous studies of erosion retardation devices, or protective structures for the lighthouse such as seawall/revetment or rehabilitation of groinfield with revetment [3]. The NPS determined that moving the lighthouse again was the most feasible, economically viable and environmentally effective plan.

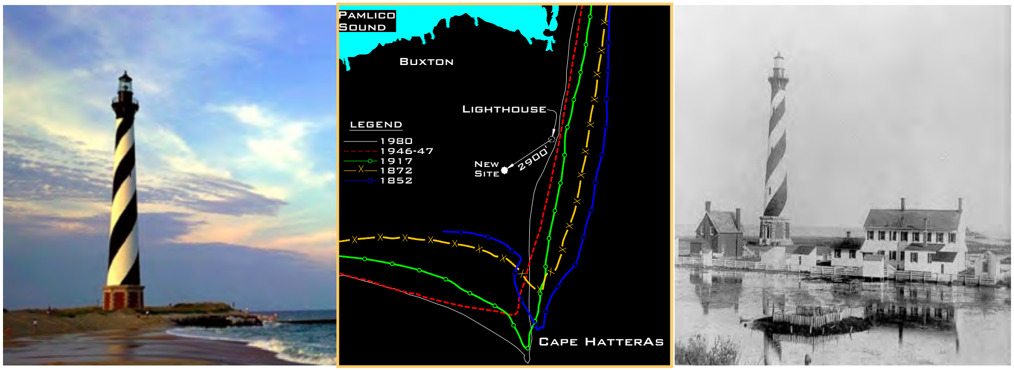

Figure 1 Shoreline regression and the flooding in 1890

2.2. Lighthouse Facts

- The lighthouse was approximately 61m (200 feet) above the ground. The estimated total weight of the lighthouse is 4,355 metric tons.
- The foundation portion of the lighthouse is composed of granite blocks that extend about 2m (6 feet) below ground. The granite blocks sit on a mat of timbers placed on medium dense to dense sand. The remainder of the lighthouse is constructed of brick with two walls and an inner gap in the lower portion, tapering to one wall in the upper portion (Figure 7)
- The ground water level is about 1m (3 feet) below the ground surface, and the water is fresh. The area between the existing location and the proposed new location consists of sands with pockets of soft organic soil. Dense sands underlie the area beginning at a depth of about 6m (20 feet).

2.3. Conceptual Design of the Move and Development of the Approach

A design-build consortium led by International Chimney Corporation and including historic architects, structural engineers, heavy moving experts, civil engineers, geotechnical and environmental engineers was selected to develop and implement the move plan. The primary requirements for the project were:

1. Move the lighthouse and adjacent structures (two houses, an oil house and three brick cisterns) to the new location and in the same spatial position.
2. Move all of the structure that is visible above the present ground.
3. Maintain the historic contexts and materials.
4. Provide for public viewing access at all times during the project.

2.4. Steps of the Lighthouse Move

In broad concept, the lighthouse move consisted of the following steps:

1. Remove the entry steps to the lighthouse and store for reassembly.
2. Replace the granite foundation with a temporary steel structure.
3. Lift the lighthouse approximately 2.1m (7 feet) vertically to align with the existing ground.
4. Use hydraulic jacks and controls to maintain the lighthouse level.
5. Construct a 7-track move support of steel laid on a prepared soil surface.
6. Use rollers and push jacks to move the lighthouse along the steel beams.
7. Remove the steel supports and lower lighthouse onto new foundation.

Figure 2 Dewatering around the foundation and cable sawing of the base

In accomplishing the move the following sub steps were necessary:

- Excavate to the base of the granite blocks (about 2m), controlling ground water by a series of pumped wells. (Figure 2)
- Separate the granite foundation by cable sawing. (Figure 2)
- Sequentially remove the granite blocks that were below original ground level while providing structural steel supports (Figure 3) with hydraulic jacks to transfer the loads of the lighthouse above to a mat of steel beams placed on the original timber mat
- Instrument the lighthouse to monitor critical stresses in brickwork and vertical inclination at all times. (Figure 3)

- Prepare move route using densification and placement of crushed stone.
- Use a hydraulic manifold system to allow multiple hydraulic jacks to be moved an equal amount or to be equally pressurized.
- After removal of the lower granite blocks, install seven main support steel beams with 100 hydraulic jacks.
- Using alternating jacking, lift the lighthouse approximately 2.1m (7 feet) above the original foundation level.
- Complete installation of steel beams to form a steel grillage that will support the lighthouse during its move.
- Layout seven steel beams as rails on top of steel mats on the prepared soil. Use rollers placed between the jacks in the main beams and the rail beams.
- Use hydraulic jacks clamped to the rail beams to push on the main beams thus moving the lighthouse.

Figure 3 Underpinning of the base and monitoring system against tilt

2.5. Steps to Preserve and Protect the Lighthouse Against Damage

Prior to starting excavation, historic architects conducted a thorough condition survey, photographing and documenting existing cracks in the structure [1,3]. Measurement devices were placed across selected cracks to allow monitoring during the move preparation and transport [6].

To allow further monitoring, a variety of other sensors were placed – temperature, wind, strain and tilt. All of the monitoring sensors fed data to an on-site computer that could also be accessed remotely (Figure3). For each parameter monitored, a pre-set warning level was programmed. If the warning level were reached, the computer sounded an air horn and dialed the emergency telephones.

Samples of brick from were tested for tensile and compressive strength to evaluate what level of stress could be tolerated. An allowable stress of about 2.75 MPa (400 psi) was established. The structural analyses also found that the lighthouse had a low center of gravity due to the heavy granite foundation. The factor of safety against overturning even under a 320 km/h (200 mph) wind, exceeded 12. The engineering team concluded that during the move preparation and transport, stresses would remain within allowable limits if the maximum tilting were less than 0.5 degrees.

The hydraulic jacking control system which could maintain constant pressure, or could produce a set amount of movement was the key to staying within the prescribed movement and stress limits during removal of the below-ground granite foundation and installation of temporary support.

Figure 4 Seven Rails, Support Frame, Jacks and Rollers for the Lighthouse

2.6. Removal from Existing Foundation, Installation of Shoring and Rails

The lighthouse foundation was detached from the lighthouse using a diamond cable saw (Figure 2). Upon completion of sawing, granite blocks were removed for placement of shoring towers for underpinning (Figure 3). Engineers estimated the weight distribution of the lighthouse and provided stressing levels for each set of shoring towers. The first granite removal left about five feet of the outer edge of the lighthouse initially supported on timber posts while the steel shoring towers were installed concurrent with granite removal. The shoring towers, consisting of four tubular steel supports each with a hydraulic jack, were placed on steel beam mats laid on the original timber foundation raft. Bearing beams were placed on the top of each tower and cushioned with timber shims against the upper lighthouse section.

Once the shoring towers were installed under the initial cut area, the jacks were pressurized to the calculated values. Then workmen removed another 1.3m (4 ft) section of the granite foundation, placed more shoring towers and pressurized them. The results from the computerized monitoring provided immediate information on changes in the lighthouse stress or tilt. If signs of movement were noted, hydraulic pressures could be increased or decreased to compensate. This phase required three months. 136 hydraulic jacks were used.

Once all shoring towers were in place, a steel grillage was installed to make a rigid platform that would support the lighthouse as it was moved. The grillage consisted of seven main beams (Figure 4) with 100 hydraulic jacks installed in pockets between the paired beams. Figure 4 shows the jack spacing. The main beams were topped with 15 single beams placed perpendicular to the main beams. Finally an exterior frame of four W36x300 beams was placed atop and tension bars were used to lock the system together.

2.7. Geotechnical Work and Improvements to the Move Route

During the above preparations, work was being done on the move route itself. Studies conducted had shown the subsurface to consist of mostly sandy soils (Figure 5). The soils were loose at the surface, but became medium dense to dense at shallow depths. Subsurface testing using the Marchetti flat plate dilatometer (DMT) and subsequent analysis indicated that only surficial densification would be needed to prepare the move route subgrade. To check the analysis, a simple surface load test was conducted using the sand-pile test concept. Settlements of the sand surface under the test pile load compared very well to predicted values [2, 4, 5]. The recommended preparation for the move route was to use a heavy vibratory compactor to densify the top 1 meter (3 feet) of sand that was above the water table and installation of 30 cm (12 in) of compacted crushed stone. Estimated settlement under the worst conditions was about 7 cm (2.5 inches) [2, 4, 5].

One unique feature of the transport section was that only enough steel mats, track beams and crushed stone to cover a portion of the total move route were purchased. During the move, these elements were removed from the rear of the move area and relocated to the front as the move progressed.

Figure 5 Geotechnical Study Along the Move Route and New Foundation

The new foundation for the lighthouse was designed as a structural concrete mat, 1.3m (4 ft) thick and heavily reinforced (Figure 5). The mat was supported on medium dense sand at a depth of 4m (12 feet) below ground level. Because the lighthouse load was to be moved horizontally across the mat to reach the final position, the structural design had to account for significant bending forces.

2.8. The Move

With the lighthouse load successfully transferred to the shoring posts and the move route preparations complete, it was time to initiate the move. First, the lighthouse had to be lifted 2 m (6ft) vertically so the base of the support system was level with the move route. The lift was accomplished using the hydraulic jacks in both the shoring towers and in the main beams. Lifts of about 30 cm (1 ft) were made, alternating between the two sets of jacks and placing wood cribbing underneath. With the hydraulic control system and the instrumentation system operating as a feedback mechanism, the lighthouse lift was accomplished without exceeding any of the stress or tilt limits.

Special low-friction roller assemblies were placed between the main beam jacks and the track steel (Figure 4). Hydraulic jacks with about 1.9 m (5 ft) extension capability were used for pushing the main beams (Figure 4). The base of each push jack was clamped to the track beam, the push arm extended causing the lighthouse to roll, the base was then unclamped, advanced about 1.9 m (5ft) and the process repeated. An average day resulted in about 40 m (130 feet) of move with the best day being 95 m (312 feet). The entire 880 m (2900 feet) was traversed in 23 days (Figure 6). Minimal weather delays occurred, and the biggest concern, hurricanes, did not materialize.

Figure 6 Moving of the Lighthouse

After final positioning on the new foundation the steel shoring towers were reinstalled and the lighthouse was lowered about 30 cm (12 in) to its final elevation. Structural brick was used to fill in the space between shoring towers, providing support as the structural steel temporary grillage was removed. Finally, after the entire open space between the concrete mat and the base of the lighthouse was filled, earth fill was placed to level the ground, the steps were reconstructed and the base of the granite foundation previously above ground surface was restored.

The lighthouse move and that of the other historic structures was a resounding success both technically, and publicly. Public viewing areas were established along the move route, and visitors could be within 20 to 30m of the move. During the project, the Park Service reported about 10,000 visitors a day with about 20,000 per day during the 23 days of the actual move.

The project was completed on time and within budget. Shortly after the lighthouse was in final position, Hurricane Dennis arrived on the Outer Banks with high winds and heavy rains. The wind sensor at the top of the lighthouse recorded 210 km/h (130 mph) before blowing away. The only damages to the lighthouse itself were a couple of broken windows.

3. CONCLUSIONS

This article discusses moving of the Historic Cape Hatteras Lighthouse out of harm's way. The project was recognized by American Society of Civil Engineers as an Award Winner for Outstanding Civil Engineering Achievement project for 2000.

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THE CONSERVATION OF THE WALL PAINTINGS OF THE HOUSE WITH IONIC CAPITALS IN HIERAPOLIS: DETERMINATION AND REMOVAL OF THE SULPHATE LAYER AND CONSOLIDATON

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ABSTRACT

The ruins of the big peristyled house, so called "The House with Ionic Capitals", are located on the road to the theater, very close to the Temple of Apollo in Hierapolis, Pamukkale, Turkey. The house decorated with wall paintings, tile and opus sectile floorings, had been the house of aristocrats since it was built in II. century A.D., damaged by the earthquakes in V. and VII. centuries A.D. and maintained being a residence until X. century [1]. This paper is aimed to discuss the conservation practices of the wall paintings of the house during the excavation season in 2005, after several years from the discovery.

1. INTRODUCTION

A noticeable whitish-grey salt deposition hiding the colors and patterns was recognised on the paintings in periodic checking in 2005. The origin of the layer and its chemical composition were unknown. This problem that arized after several years from the discovery was the main factor for the intervention including identification of the salts and their removal using suitable, undestructive materials and methods, and consolidation.

2. PRODUCTION TECHNICS, PREVIOUS INTERVENTIONS AND STATE OF CONSERVATION

It was observed that not a uniform technic, but some variations of technics were used to prepare intonaco. For instance, there were two layers overlapping in some areas, however observations gave the impression that pigments were mostly applied on dry plaster like secco technic. The efflorences caused decay of the paint partly resulting local disintegrations and detached some of these overlapping paint layers instead of effecting the visual perception negatively.

The wall paintings were cleaned and consolidated by plaster mostly and by synthetic resin limitedly during the previous conservation processes. It was reported that appropriate conservation material was used, but harmony between the original colors and the color of consolidation plasters was not considered (Figure: 1). Besides some adobe mortar was applied limitedly to a small area which should be seperated and substituted with lime bindered plaster.

Figure 1 Efflorences hiding the patterns and the colors and the white colored plaster pointings (on the upper part on the left side).

A protective roof was built and a wooden barrier was constructed in front of the walls to prevent mechanical deterioration. The barrier was located 20 cm away from the wall paintings and they were covered with appropriate permeable synthetic textile enabling air circulation to ensure evaporation in case of water accumulation by capilary absorbtion or infiltration.

3. IDENTIFICATION OF THE LAYER

Simple qualitative spot tests were preffered because they were low costed and available to realize in the conservation laboratory of the excavation. Finding out the presence of chlorides, sulphates, carbonates and nitrates was aimed beause they are the major probable water-soluble anions existing on site. 11 samples were taken from different zones where colors and tones vary.

Test Procedure

1. Samples were pulverized in ceramic muller and 100 mg of each were treated with deionised water, warmed up to 60ºC and left for two hours to obtain stock

solutions. Almost no undissolved particles were observed to filter. Some deionized water was stocked and every test was repeated also for it to be able to compare their blurriness and colors to the colors obtained from the sample solutions.
2. Chloride Test: 2 drops of dilute nitric acid (HNO_3-2N) and 2 drops of dilute solution of silver nitrate ($AgNO_3$-0,1N) were added to the sample solutions. The mixture was observed for a whitish, gelatinous precipitate of silver chloride (AgCl) indicating the presence of chlorides.
3. Sulphate and Carbonate Tests: 2 drops of 10% solution of barium chloride ($BaCl_2$) were added to the sample solutions. The mixture was observed for a white precipitate of barium sulphate ($BaSO_4$) or barium carbonate ($BaCO_3$). When precipitation was observed, a drop of concentrated hydrochloric acid (HCL) was added. No change indicated the presence of sulphates whereas color change, dissappearance of blurriness or releasing of bubbles of carbon dioxide (CO_2) indicated the presence of carbonates.
4. Nitrate Test: A small diphenylamine crystal was put on a glass plate and mixed with 2 drops of sample solution. After drying, a drop of concentrated sulphuric acid was added and observed for a blue color indicating the presence of nitrates.

The results showed that the main constituents were sulphate salts. Little quantities of chlorides and nitrates were detected. Table showing the results of the tests is given below.

Table 1

Sample No	Cl^-	SO_4^{-2}	CO_3^{-2}	NO_3^-
1	+-	++	+-	-
2	-	++	-	+
3	+	++	-	+
4	-	++	-	-
5	-	++	-	+
6	+	++	-	-
7	-	+++	-	+
8	+	+++	-	+
9	-	+++	-	+
10	+-	+	-	-
11	+-	++	-	-

- =Absence of the anion
+- =Limited concentration of the anion
+ =Presence of the anion
++ =Presence of the anion in significant quantity
+++ =Presence of the anion as a main component

4. CLEANING

Removing the chlorides and nitates by a water containing compress is feasible as they are water soluble in general. But a compress of deionised water and Arbocel (paper pulp) was not capable of removing the whole layer, indicated the probable presence of slightly water soluble calcium sulphate. In this case, two alternatives were thought to remove the layer. These were; atomized water cleaning or chemical cleaning using a buffer solution. As atomized water method would take longer time, required equipment that was not available and required a big amount of water (tap water in the village was very rich of sulphates and chlorides), a weak basic buffer solution called AB57 was preffered.

Because there are nuences in preparation of the mixture in conservation literature depending on the dirt's and the artefact's nature, several tests were carried out in 2 cm^2 sized areas with Mora's classical formulation for wall paintings [3] and also with an alternative modified formulation. Mora's classical formulation was decided to be modified by replacing sodium bicarbonate with ammonium carbonate, addition of a chelator EDTA (disodium salt of ethylen diamine tetra acetic acid) as prefered for calcerous stone cleaning formula of AB57 [4] and addition of Arbocel instead of CMC (carboxymethyl cellulose). The reasons for modification are listed below:

1. There was the risk of some sodium bicarbonate residues remaining after application that may efflorence continuously in the future, whereas ammonium carbonate and ammonium bicarbonate residues decompose to form carbondioxide, ammonia and water in the course of time. Therefore ammonium carbonate was prefered to achieve the buffer.
2. A chelating agent disodium EDTA was used to increase the solubility of probable calcium sulphate, so the cleaning operation would require less time and therefore the paint would be in less contact with the solution.
3. Removing the poultice is easier from the rough surface of the wall painting if Arbocel is used as a thixotrophic substance whereas CMC requires a japanese paper layer as a seperator.
4. "Desogen" (Geigy), is not produced anymore so another nonionic surfactant "New Desogen" was prefered instead.

Here below are the Mora's original formulation for wall paintings and the modified formulation.

Mora's formulation for wall paintings:		**Modified formulation:**	
Water	1000 ml	Water	1000 ml
Ammonium bicarbonate	30 g	Ammonium bicarbonate	50 g
Sodium bicarbonate	50 g	Ammonium carbonate	30 g
"Desogen" (Geigy) of		Disodium salt of EDTA	25 g
10% strength	25 g	New Desogen 10% strength	10 ml
CMC	60 g	Arbocel was added until a dense poultice was achieved	

The modified formulation showed better results, so it was tested 3, 5 and 10 minutes-long to determine the application time. 3 minutes test was found inedaquate and finally, 5 minutes-long application was preffered as some red color was detected on the poultice in 10 minutes-long test. In the case of total removal of the salts that were crystallized between the paint layer and intonaco, some paint could be washed off. In this respect double application of the compress (poultice) for 3 minutes was not prefered. The steps below were put into practice for the cleaning operation:

1. Dry cleaning by soft hair brushes.
2. Preconsolidation of disintegrated plaster and the zones of the paint which had the risk of removal by means of brushing or injecting 3-5% (w/v) Paraloid B72, a reversible ethyl metacrylate-methyl acrylate copolymer, and also by means of squeezing with spatula over japanese paper compress (Figure: 2).
3. Cleaning with water spray and soft hair brushes.
4. Saturation of the wall painting with water to minimize the absorbtion of the solution therefore it can contact much more to the target area where the sulphate efflorences existed.
5. Saturation of the solution with Arbocel and covering a reasonable zone of the wall painting surface quickly (Figure: 3).
6. Removal of the poultice five minutes later from the application.
7. Cleaning with hair brushes and water spray in order to increase the effect by means of mechanical action together with chemical impact of the solution.
8. Covering the washed surface by dampened Arbocel compress to absorb the probable residues of the dissolved salts and the cleaning solution.

Figure 2 . Consolidation of detached overlapping layers by injecting 5% Paraloid B72 in acetone and squeezing with spatula over japanese paper compress.

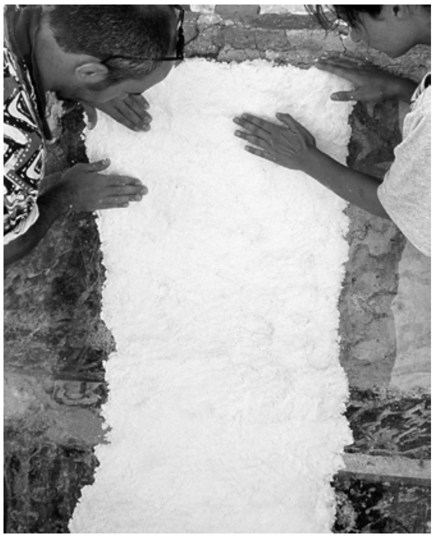

Figure 3.Covering the painting surface with AB57 compress.

5. CONSOLIDATION

Before the consolidation plaster entegrated to the wall painting, parts which had risk of falling were reinforced temporarily by japanese paper and %10 (w/v) concentration of Paraloid B72, dissolved in acetone. On the other hand, zones that were risky to clean without consolidation were first consolidated with plaster and then cleaned at the end of the schedule so that enough time was achieved for carbonation process and other reactions that form calcium silicates.

The plaster was produced with lime as binder, river sand, limestone powder, travertine powder as agregates in respect of 1:3 (binder:agregate) ratio by volume. Also brick powder was added as an artificial puzzolanic agregate to achieve hydrolic properties, but was not included considering the harmony of the original colors when the visible parts like edges were being reinforced. According to the observations on the original plaster, larger sized agregates sieved for not reducing the porousity. Therefore, obtaining the crystalisation on the consolidation mortar rather than the original plaster was aimed in the case of efflorence repetition. Also, achieving good adhesion, cohesion and enough hardness, but no more than the original, were considered. However, in case of narrow cracks, very fine aggregates was sieved and sand was not added in order to inject the plaster easily (Figure: 4). Injection plaster included more water and an acrylic resin Primal AC 33 3% by volume as a protective colloid. Before plaster injections took place, deteriorated plasters were vacumed and after the mixture of ethyl alcohol and water in respect of 1:2 ratio was enjected to remove the residues and wetten the canals. Then Primal was injected in %10 concentration in water to reduce the absorbtion of the water of the plaster by the wall.

To stabilize the pigments and to fix some loosen parts of intonaco, 3-5 % (w/v) concentrations of Paraloid B72 in toluene and 10% (v/v) concentration of Primal AC33 acrylic resin in water were applied very limitedly by brushing, spreying and injecting where necessary. Toluene was preffered as the solvent where deep penetration was desired while acetone was favoured to achieve rapid adhesion.

Figure 4 Injection of the plaster from the hole whose sides were reinforced with japanese paper and Paraloid B72 to prevent cracking.

6. MODIFICATION OF THE ROOF

Present protective structure was enlarged, its inclination was arranged and a pipe was added to carry out the accumulated water away from the wall into the ancient sewerage system (Figure: 5). The pedestries of the roof containing portland cement filling was removed and replaced by a new mortar including slaked lime

(1 part), white cement (1 part) as binders and sand (3 parts), limestone (3 parts) as agregates and gravel.

Figure 5. The modified roof.

7. CONCLUSIONS

A few days after the cleaning procedures were completed and the wall paintings were dried entirely, it was recognized that the salt layer on the surface was removed reasonably so the patterns and colors were seen much more clear (Figure: 6). Properties like texture, color, porosity, adhesion, cohesion, and hardness of the consolidation plaster was satisfactory.

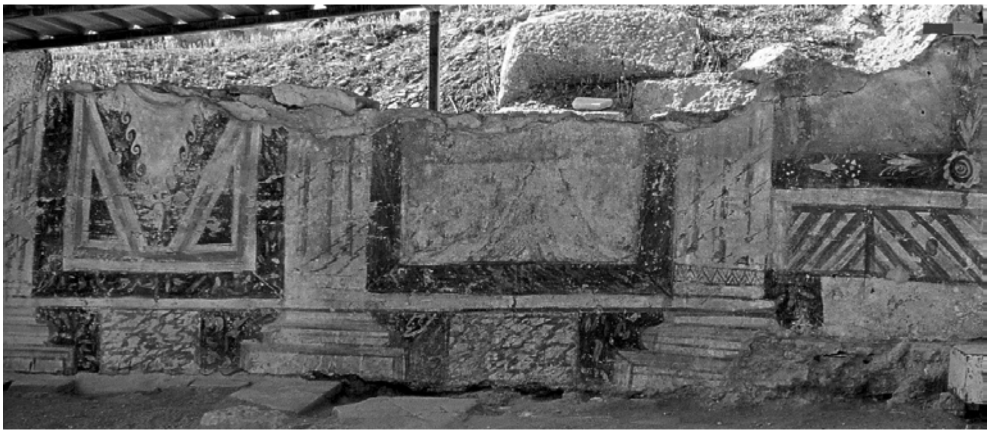

Figure 6. Photograph taken after entire drying.

The goal for the modification of the roof was to lower the rain water infiltration. Minimizing the probable sulphate releasition was aimed by reducing the concentration and changing the type of the cement in the pedestals. In order to test the modified roof, water was spilled over and no infiltration was observed, and the pipe worked well. In the future, unless water accession stops, designing a larger structure will be necessary. Furthermore, the ground soil should be analysed to determine its sulphate content.

The whole painting surface was not treated with synthetic resins considering the risk of water reach to the walls causing the dissolution of the sulphates again. In this circumstance, under-surface crystallization would occur between resin layer and the plaster instead of surface crystallization, and therefore, their removal would be harder and might cause certain harmful results. If sulfatisation stops, application of acrylic resin layer to the whole surface may be put on the agenda to prevent the harmful effects of condensation and high relative humidity on the pigments and salts. Besides, a glossy effect for a better visual perception of colors and patterns will be obtained.

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DESIGN OF HYDRAULIC GROUT AND APPLICATION METHODOLOGY FOR STONE MASONRY STRUCTURES BEARING MOSAICS AND MURAL PAINTINGS: THE CASE OF THE KATHOLIKON OF DAFNI MONASTERY

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ABSTRACT

In the present paper, information is provided regarding the selection of grout applied in the Katholikon of Dafni Monastery and the methodology of its application, as well. The design of high injectability grouts was performed on the basis of the demands derived from the structural restoration project of Katholikon of Dafni Monastery. In order to obtain data regarding the mechanical properties of masonry, six three-leaf stone wallettes bearing mosaics, were constructed and subjected to mechanical tests before and after the application of the selected grouts. These experiments were decisive both for the selection and the application of the grout.

1. INTRODUCTION

This paper is in direct conjunction to papers [1], [2] and [3]. In order to obtain data regarding the mechanical properties of masonry in Dafni Monastery large scale specimens were constructed and tested: (a) twenty eight cylindrical specimens (which simulate the infill of three leaf stone masonry) strengthened by different grout formulations subjected to compression [3] and (b) six three-leaf stone wallettes, bearing mosaics on their façade, subjected either to compression or to diagonal compression up to their maximum resistance. The wallettes were tested before and after grouting with two alternative grouts developed on purpose, in order to select the most appropriate proportions for application in the Katholikon of Dafni Monastery [1, 2].

The design of high injectability grouts was performed on the basis of the following performance demands [4]: (a) the necessary strength of the grout which ensures satisfactory strengthening of the masonry walls was roughly estimated to 6-10MPa for compression and >2MPa for flexion, at the age of 6 months, predicting a strengthening of the wall of the order of 100%. This prediction was made via empirical expressions mentioned in the literature [5, 6], (b) the appropriate physico-chemical characteristics of the raw materials so that to avoid jeopardising the durability of the structure and its mosaics, (c) the necessary injectability of the grout to enter and fill the voids and cracks; to this end the nominal minimum width of voids and fissures to be injected (W_{nom}) was set to be equal ~200μm.

Based on previous investigations carried out from the Directorate for Technical Research on Restoration (DTRR) and also from the pertinent bibliography [7-9], two main categories of hydraulic grouts could satisfy both injectability and strength requirements: (i) a ternary grout composed of white cement, lime and pozzolan and (ii) a hydraulic lime – based grout, as the hydraulic lime has the advantage of being most compatible with the original materials of the existing masonry, while offering the hydraulic properties to give an acceptable early strength.

On the basis of the results of the whole experimental project [2, 3], it was decided to use the hydraulic lime based grout in the Katholikon of Dafni Monastery. The substantial (compressive and tensile) strength enhancement of wallettes, the rather ductile behaviour under diagonal compression (compared to that of masonry grouted with the ternary grout), the physico-chemical properties that ensure a durable intervention and contribute to the protection of mosaics led to the selection of a hydraulic lime based grout. In order to improve the grout formulation the addition of a small percentage of pozzolan (10%) was decided based on additional data deriving from porosity measurements, salt durability tests and from in situ trials of the grout formulations.

In the present paper the design of the grouts will be presented, along with the methodology to be used for the application of the injections on the monument.

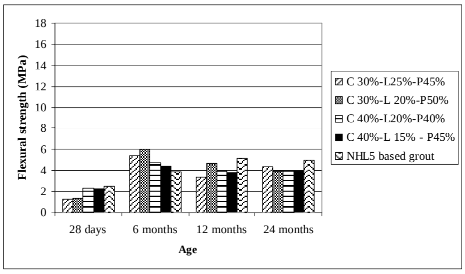
2. SELECTION OF GROUT PROPORTIONS

Taking into account the results of DTRR's previous investigations (Figure 1), a ternary grout having a solid phase composed of a low cement content (30%), lime 25% and pozzolan 45%, could fulfill the aforementioned strength requirements. In this research the use of white Danish cement in the ternary grout was chosen, due to its fineness, low alkali content and high sulphate resistance.

Concerning the hydraulic lime – based grout, various types of natural hydraulic limes (classified by EN 459 as NHL 2, NHL 3.5, NHL 3.5-Z and NHL 5), were tested in the laboratory of DTRR, and various grout formulations with or without superplastizer, were examined. The main results of the research are represented in [3].

In order to determine the optimum percentage of water series of grouts were prepared and tested. To this purpose the standardized sand column test method (NF P18-891, pr. EN 1771), was used to check the penetrability and fluidity along with the standard apparatus for testing the fluidity (NF P18-358) and stability (NF P18-359) of the suspensions. In each case, a time limit of 50 sec was set for the sand column penetrability test (T_{36}); an efflux time of 500ml of grout less than 45 sec (Marsh cone d=4.7mm fluidity test) and fluidity factor higher than $1.5x10^3$mm/sec [10], as well as a maximum acceptable limit of 5% for the bleeding test [3, 5].

After comparative evaluation of the results, two grout compositions (the ternary grout and a natural hydraulic lime (NHL5)-based grout) fulfilled simultaneously the injectability, the strength and durability requirements [3] and therefore were selected to be applied to the aforementioned wallettes [2]. From the outcomings of this project (see Paragraph 1), it was decided to use the hydraulic lime based grout in the Katholikon of Dafni Monastery.



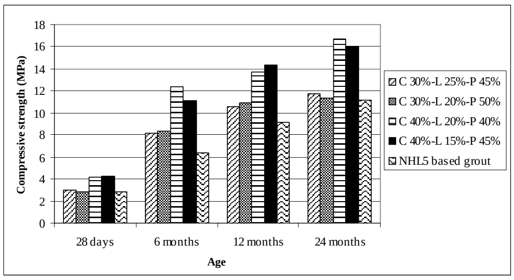


Figure 1: Evolution of strength of grouts injectable through sand column 1.25/2.5mm (T_{36}<50sec) which simulates voids and cracks of 0.2-0.4mm (W_{nom}~200μm). Water /Solids= 0.8, Superplasticizer 1% of the solid phase. (C=white cement, L=lime, P=Pozzolan, NHL5=natural hydraulic lime), (DTRR)

Given that these hydraulic limes are characterized by a relatively high percentage of available lime, the addition of pozzolan, in adequate proportion, is expected to have beneficial effect. Thus, in order to improve the hydraulic lime based grout, the addition of fine natural pozzolan (dmax<75μm) in various proportions was investigated. Tables 2-4 summarize the main experimental results.

The grouts were prepared using raw materials from the worksite and a high turbulence lab-mixer (at 10000rpm), which simulates better the in situ mixing conditions, for this type of grouts. The mixing time for grout A1 was 3 minutes and for A2, A3 grouts, 4 minutes (2min/solid component), respectively.

Table 1: Chemical and physical characteristics of pozzolan used

Soluble ions (%) ASTM C114-03 and IC	**Na^+**	**K^+**	**Mg^{+2}**	**Ca^{+2}**	**Cl^-**	**NO_3^-**	**SO_4^{-2}**
Pozzolan	0.0853	0.0182	0.0069	0.0325	0.1001	0.0074	0.0696
	d_{85}	**Average grain size**		**Reactive silicon dioxide (% w/w)**		**Pozzolanicity Index**	
Pozzolan	30.5 μm	9.5μm		>60%		7.5 N/mm^2	

Table 2: Injectability characteristics of NHL5/pozzolan grouts

GROUT	**A1**	**A2**	**A3**
NHL5 (St Astier)	100%	90%	80%
Pozzolan		10%	20%
Superplasticizer *	1% (SP1)	1% (SP2)	1,5% (SP2)
Water *	80%	80%	80%
Bleeding	<1%	<1%	1%
T36 (sec) – Sand column 1.25/2.50 mm (voids ~0.2-0.4mm)	23	19	36
Fluidity factor (x10^{3}mm/sec)	2.24	2.13	1.86
Apparent viscosity (sec) - Marsh cone d=4.7mm - 500ml of grout			
0 min after mixing	20	21	23
60 min after mixing (agitated)	26	23	27
* % of the solid phase of the grout. SP1 and SP2 superplasticizers based on lignonaphthalene salts and polycarboxylic ether, respectively			

Table 3: Flexural and compressive strengths of NHL5/pozzolan grouts

GROUT	Compressive and flexural strength (MPa)					
	Age (days)					
	28		90		180	
	f_{gc}	f_{gt}	f_{gc}	f_{gt}	f_{gc}	f_{gt}
A1	2.01	1.16	4.50	2.52	5.97	2.09
A2	2.51	1.41	5.26	2.80	6.04	2.44
A3	2.11	1.22	4.05	1.67	6.54	1.29

Salt durability tests were carried out on specimens of grouts A1, A2 and A3, at the age of 12 months following a procedure described in [3].

After the comparative evaluation of the results, on the basis of the injectability of the grouts and its preservation in time, the evolution of the grouts strength and their behavior to salt decay, the grout composition A2 (90% NHL5/ 10% pozzolan/ 1% SP2/ 80% water) was selected to be applied.

Table 4: Mass changes (%) and damage pattern during salt durability tests

GROUT	A1	A2	A3
Mass changes until the 6th cycle Drying at 20°C and high RH. (up to)	21.1%	13.2%	12%
Mass changes from 7th to 11th cycle. (up to)	-2% (edge rounding)	-2.2% (edge rounding)	-4% (delamination)
Mass changes from 7th to 9th cycle. Drying at 50 °C and low RH. (up to)	-4.4% (edge rounding)	-5.3% (limited delamination - edge rounding)	-8.6% (delamination - edge rounding)

3. GROUT APPLICATION

3.1. Wall preparation

The preparation of the aforementioned wallettes before grouting, outlined the methodology followed in the case of Katholikon of Dafni Monastery. That includes: (a) Drilling of injection holes, long enough to access the infill of the wall. The holes were drilled mainly at the joints and followed a grid, b) Plastic tubes of 10mm diameter were placed inclined downwards into the injection holes at the two facades of the walls. Furthermore, tubes of smaller diameters (2 - 8mm) were placed on mosaics at depths of 3cm and 1.5cm in order to reach and control the grout flow behind the substrata and tesserae, respectively. The plastic tubes were marked with tapes of different colour so that to recognize the depth of different tubes and numbered from the bottom to the top. Marking the tubes enables the record of entrances and exits of the grout and makes easier the understanding of injection process, c) Sealing of the cracks with the appropriate mortar to prevent the leakages.

The in situ preparation of the Katholikon masonry included also the careful removal of recent repairing mortars, thus revealing and preserving the underlying byzantine ones (Figure 2).

Figure 2: Wallette preparation for grouting (left). Removal of the recent repairing mortars in order to reveal the underlying byzantine mortars of the Katholikon masonry (right)

Figure 3: Preparation of the Katholikon masonry and its mosaics before grouting

3.2. Grout Injection

Mixing of grouts, for the pilot applications, was achieved with a prototype ultrasound dispersion mixer of 20 lts capacity (28KHz, 1,2KW, 220V), assisted by a mechanical device of low turbulence (300rpm). After mixing, the batches of grout were drained into an air-proof cylindrical collector of 40 lts capacity, equipped with a magnetic agitation system. The cylindrical vessel was made of Plexiglas to enable the level observation of the grout. Into this vessel compressed air has been supplied (1bar), pressing uniformly the surface of the grout and forcing it to exit from a long pipe ended at the bottom of the vessel.

Consequently, the fluid was introduced into the wallette through a flexible pipe bearing at its end a nozzle of small diameter equipped with a manometer. All the aforementioned devices were especially manufactured for pilot applications. Similar equipment will be used for the grout application on the mosaics of Katholikon, while for the in situ masonry grouting the use of industrial equipment (high turbulence colloidal mixer, agitator, pump, recorder of grout flow and pressure, nozzle equipped with manometer etc) was necessary. Before injection started, tests took place in order to check the whole injection system and the quality of the grout.

The grout was injected under low pressure (around 0.75 bar, controlled by the manometer at the end of the pipe), starting from the bottom and following a perimetric path to the top of the wallette in order to ensure that the grout was rising up and filling the internal part of the wall forced by pressure. At the same time there was a record of entrances and exits of the grout, as well of its volume consumed in each entrance. Those two factors gave important information about the movement and the effective performance of the grout. In the case of the wallettes, the injection work was usually completed pumping the grout from one or two tubes located at the bottom; and the consumed quantity of the grout was almost similar and in good agreement with the calculated amount of the voids from the construction of the walls (~40% voids on the volume of the infill or ~10% on the whole wall volume). Thus, a complete filling of the voids was assumed.

Figure 4: Equipment used for the pilot grout applications

At the worksite all the aforementioned data (entrances and exits of the grout, volume consumed per each entrance, pressure fluctuations etc) were also fully recorded (on diary cards and drawings, see Figure 5), as well as daily measurements of the quality characteristics of the grout were taking place.

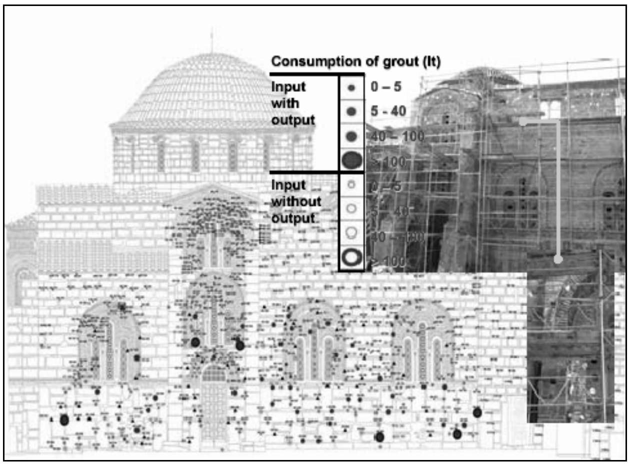


Figure 5: Recording of the injected quantities of grout (per each entrance) on drawings

CONCLUSIONS

Both the ternary and the hydraulic lime based grout proved to have high injectability as they filled the voids and cracks of the wallettes in a satisfactory way. In fact the predicted volume of voids was measured to be filled by the grout. The methodology developed for the application of grouting permitted the preparation of the wallettes without taking away the mortars, while in the surface with the mosaics, fine tubes were installed to permit the grout exit. Furthermore, the cautious recording of all the relevant data permitted the continuous monitoring of grouting.

The whole procedure served as a rehearsal for the application of the grouting in situ, not only for the control of the quality of the grout but also for the monitoring of grout movement in the walls. Thus a specific methodology was designed for ensuring the quality of the in situ application of the technique.

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HISTORICAL KONJIC BRIDGE / BOSNIA AND HERZOGOVINA

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ABSTRACT

Konjic Bridge which is situated in the City of Konjic on km 52 of Sarejovo – Mostar State Road and over the River Neretva, was constructed by Mehmet IV, the Sultan, (Hunter Mehmet) during his reign in 1682. The Bridge was substantially damaged during World War II as a result of the bombards by the German infantry by the time of their withdrawal. The application projects of the historical Konjic Bridge the reconstruction of which is committed by the Government of the Republic of Turkey, furnished by the Division Directorate of Historical Bridges serving as consultant in the preparation of measured drawings and preliminary design of the restitution and reconstruction works has been awarded the success reward by the Chamber of Architects due to **‘***The claims for a historical structure out of Turkey, in public level and this particular project raises hopes for the preservation of the historical bridges within Turkey; the decisions for the protection, repair and reconstruction of the bridge is based on intense scientific and technical investigations and particularly to the success achieved in the correct integration of the preservation techniques together with the engineering knowledge.*’ (X. National Architecture Exhibition and Rewards)

Referring to the application project developed specifically for the restoration of the Konjic Bridge the construction supervision of which is currently being conducted by our Division Directorate, this particular declaration is aimed to serve as an access in international platform the decision taken for the preservation, restoration, repair and reconstruction processes together with the recent techniques and transfer all such data to the next generations.

1. HISTORY OF THE REGION and THE BRIDGE

Bosnia and Herzegovina was annexed to the Turkish authorization by Fatih Sultan Mehmet Khan in 1463.

Out of 271 each Turkish bridges in the territory of Balkans, 121 each of such bridges are in Bosnia and Herzegovina. The Konjic Bridge which is situated on River Neretva in the City of Konjic was a strategically important bridge due to the fact that it was the only passage on the river during the period it was first constructed. [9]

Before constructing today's Konjic Bridge, there constructed several bridges on River Neretva. In 1665, Evliya Çelebi who visited Konjic then, specified that there was a large Wooden Bridge on River Neretva which is referred to as Karagöz Bey Bridge. On the inscription of the subsequently constructed stone bridge it is stated that the bridge was constructed in Hegira 1093 (Gregorian 1682). In addition, it was also stated therein that the old wooden bridge was destroyed in 1666 and the construction of the new bridge lasted three years. [2]

The Sultan Mehmet IV issued a decree for the repair of Konjic Bridge. The said decree was found among the Eastern Collection of Herzegovina Franciscan by Hıvzija Hasandediç.

The Konjic Bridge is a perfect example of classical Ottoman Bridge of 17th century. The Bridge is double centered with six arch spans in ogival form. The form of the arches, inscription pavilion, flood damper and silhouette are similar with the Bridge Drina constructed by Architect Sinan in Visegrad upon order by Sokollu Mehmet Pasha. (Figure 1)

The bridge was substantially damaged as a result of the bombardments on March 03, 1945 during World War II. (Figure 2)

Fig. 1: The bridge before its demolition

Fig. 2: The bridge after bombardment

2. CURRENT STATUS

The sound arches left after the bombardments were demolished during construction of the wooden bridge to connect both of the ends on River Neretva. Soon after determination of the decays on the wooden bridge serving for the pedestrian crossing, concrete caps were placed on the remaining abutments of the mason bridge. The bridge was opened to the traffic through the reinforced concrete flooring poured over the steel girders placed on such caps The concrete caps placed onto the stone abutments damaged the masonry material.

The bridge with spans varying from 13.50 m to 6.80 m and 6 arches is 86.20 m length and 5.35 m width. The bridge arches are settled on five each stone abutment placed onto the wooden girders.

It has been observed on the wooden grates that there occurred decays, ruptures and discharges on the wooden grates due to the changes in the flow regime in the river bed and to the recessions around the abutment foundations. (Figure 3)

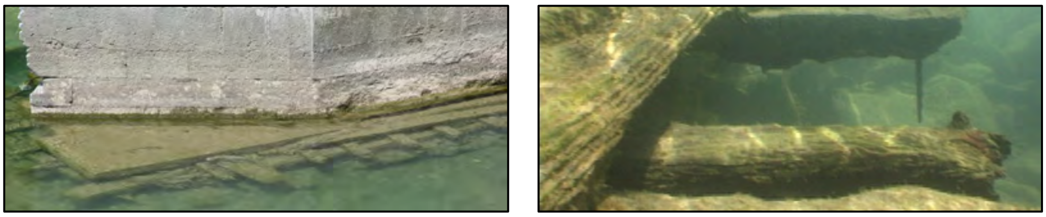


Figure 3: The wooden grates forming the foundation of the bridge abutments.

Two different types of stones are used in the bridge. The initial point of the arches on the original abutments, the materials in flood damper, upper cornices of the hinge are made up of solid stone starting from the springing level up to the two stone rows while the remaining section is made up of soft stone. On the surface of the stones there observed vegetation and floral dispositions as a natural result of vegetation.

2.1 Erosion and recession around the foundation

The pointed sections of the flood dampers to the upstream level are subject to refraction due to the strokes of the materials such as trees, logs, stone etc carried away by the floods. In particular, the damages arising out of intense water flow around the pointed edge of the abutments can best be observed. To the upstream, the sand pit which is 400 meters away from the bridge covers an area of 250 m x 60 m and it gives way to water flow width limited with an area of 20 meters which in turn accelerates the movement of the compressed water. The water restricted in area of 30 meters which is 200 m distance to the bridge with over flow regime carries all stream material in the river bed. The foundation base of B3 abutment is particularly under risk due to the fact that the said abutment is close to the sand pit and that it has enormous sand reserves. (Figure 4)

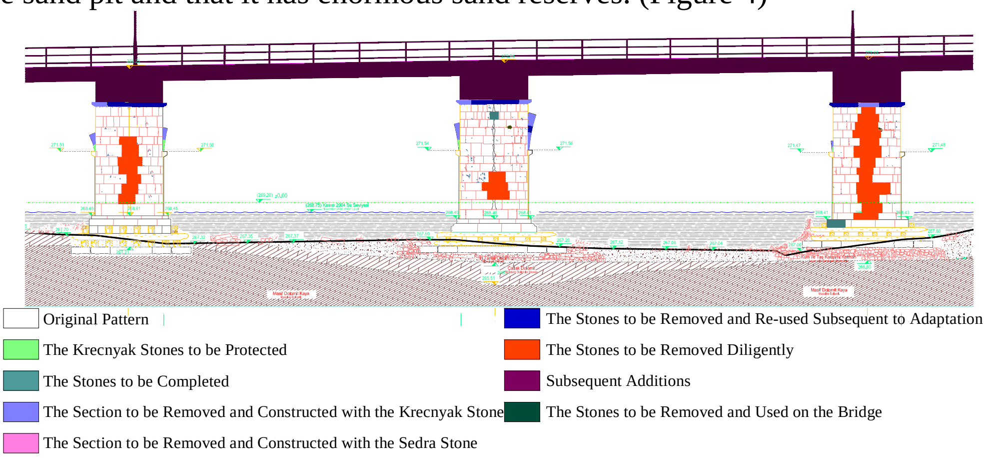


Figure 4: Upstream – Measured Drawing Analysis

In addition, the wooden grates forming the foundations are obviously exposed due to the stream materials carried away through the decrease in the water level on river foundations.

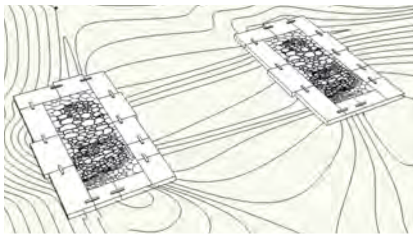
The decays on the woods are due to the integration of water with air, that is to say, dry and wet conditions thereto (oxidation) and in practice, the continuous aquatic conditions do not give way to such a case. [3]

2.2 Wooden Grate System

The median stone abutments of the bridge are settled onto wooden grates in two or three rows. The sectional dimensions of the wooden girders are 20x20 or 30x30 cm and each row contain 6-8 and 15-21 grates. The length of the same varies between 5 meters and 11 meters. (Figure 3)

The wooden grates are placed on a stone platform with blockage fills which function as pads and which is made up of dolomite –limestone type local rock formation at a height of 30-35 cm. That is to say that, a flexible foundation base is in question. This can be resembled to elastometer support and seismic isolation which in general terms is used to strengthen the structures against earthquake. The aim of the seismic isolation method is to place flexible energy oscillator between the foundation and the bases of the structure and thus decrease the seismic forces transferred to the structure from the foundation. The bearing system in historical bridges gives way to compression. The horizontal loads such as water flow, earthquake etc., cause both pressure and tension strength. Thus during restoration, the flexible foundation connection system which is also referred to as original structure shall be kept in its position and rigidities through injection and concrete casing shall be avoided. [3,6,10]

3. ORIGINAL CONSTRUCTION TECHNIQUE OF THE BRIDGE



1. The platform established through the stones clamped with each other through blockaded fills in order to form the foundation.

2. The wooden grate in the first row

3. The wooden grate in the second row and the fills inside.

4. Ampatment and initial abutments

5. Abutments up to springing level

6. Construction of the arch and the cornices

7. Construction of the flood damper and the hinges

8. Construction of side walls, vault and interior fills

9. Completion of the vault after assembly of the key stone

10. Siding and fillings under the floor

11. Construction of cornices on the sides

12. Construction of parapets

13. Construction of floors

4. RESTORATION WORKS

Any monument is an integral part of the history it witnessed and its environment. The aim of restoration is to preserve and make clear the aesthetic and historical value of the monument. Therefore, before commencement to the design drawing of the historical Konjic Bridge, a detailed archive survey was carried out [4] to collect data for the restitution project and basic data which may direct the restoration process were collected through the documents such as satellite viewings, previously issued measured drawings and restoration projects, previously taken photographs, maps etc. Apart from these, cooperation was made with the other entities and organizations to carry out the works in the most scientific and correct manner. [3,4,6,7,10]

The application projects are drawn subsequent to evaluation of the current status of the bridge in accordance with the topographical figures submitted to our Division Directorate by the Municipality of Konjic and the design drawings [1] for Konjic Bridge produced by Dzemal Çeliç.

In addition, the original core samples are taken from the bridge abutments upon execution of drilling works in the said bridge by the University of Dzemal

Biyediç and these are submitted to our Division Directorate together with the data for the maximum – minimum water levels and flows in the river. [2]

The analysis of the original grout brought from the Bridge of Konjic was made by the Central Restoration and Conservation Laboratory [7] in Istanbul and the compression and bending of tension tests of the grout prepared in accordance with the same mix ratios were carried out in the material laboratory in the Department Head of Technical Research of our Directorate General.

The deficient sections are completed and harmonized with overall structure while the parapet, cornice, flooring and pavement stones taken from the stream bed are analyzed and measured. Upon completion of the above mentioned works, the application projects are produced as per the data obtained.

4.1. The restoration techniques to be applied;

1. The concrete caps shall be extracted with due care and diligence in order to avoid any damage to the stone abutments (Figure 5).

Figure 5: Extraction works on bridge abutments (2006)

2. On the measured drawing, the arch with closed niche shall be opened to allow uniform flow of water in all niches through five spans of the six spanned bridge.
3. It was recommended to remove the hot water pipes passing through the abutments of the bridge.
4. The damaged, decayed and broken woods shall be replaced with new wood pieces in the manner not to damage the original elements. (Figure 6)

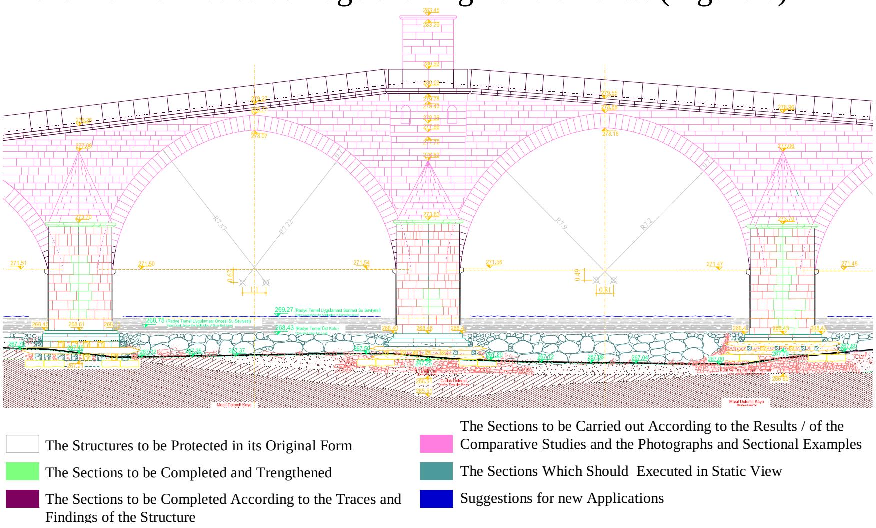


Figure 6: Upstream- Restoration Analysis

5. There exists springing line in all arches. The stones are present at the initial rows (varies between 2 to 4 rows). This gives us an idea about the original form of the arches. All of the arches are double centered. In addition, previously taken photographs are used and the distance between the bridge abutments is used for horizontal values while flood damper cornices and the existing vault arches are used for the vertical values through interpolation method.
6. In order to have an idea about the bridge, the previously taken photographs were scanned. In addition, for the Inscription Pavilionthe Drina Bridge in Visegrad was scrutinized and the bridges constructed in 17th century were compared with each other.
7. The materials in the stream bed are dragged through ever increasing flow of the undulated movement of the water which has limited area of flow around bridge niches where the width of passageway is narrowed due to the soil fill which was made in order to have some earth on river banks and to the randomly taken material from the stream bed. Thus the abutment foundations are all exposed.

In order to overcome such adverse conditions, it was recommended to the Municipality to move the sand pit which narrows the width of water flow backwards.

In addition, the ground level shall be brought to its original position by means of stacked stone fortification which is to cover the bridge foundations made up of loose material with weak bearing capacity. Thus water shall flow uniformly in all niches.

The dimensions of the stones to be used for fortification purposes with natural materials should range between (0,50 – 1,00 m) and placed with due care. (Figure 7 and 8)

It is a must to preserve this foundation structure whose soundness and reliability are thus proved together with its constituent original material not only in view of witnessing a historical event and a civilization but also of the fact that the witnessed history is an integral part of the preserved environment. The wooden grates should be definitely preserved.

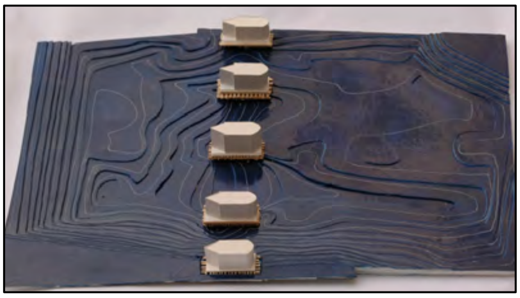


Figure 7: The status of the land where the bridge abutments are settled.

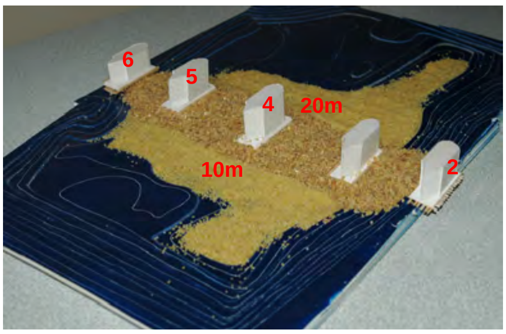


Figure 8: Model stone fortification

5. RESULT

The historical bridges shaped by the natural obstructions are plain but functional structures planned for transportation purposes from one bank to the other. The foundation systems of many of the bridges dated back to Seljuk and Ottoman periods are built on the wooden piles where the ground is made of materials the bearing capacity of which is weak along with the loose materials and on wooden grates where the ground is solid. In each case, with the introduction of the system the validity of which is proven over the years, an "elastomer support" was formed and the seismic forces transferred from the ground to the structure were decreased by placing oscillation elements between the ground and the foundation of the structure. The long lasting soundness of the bridge has proven the verification of this particular solution. (Figures 14 and 18)

The historical bridges which provide passageway for heavy vehicles today should, in the first priority, be stabilized in order to allow for safe passage and the effects which disturb stability should be eliminated in the course of restoration works while the work should be strengthened and the materials considered to be used should be selected depending on the original construction technique. Meanwhile, preservation of the wooden foundation structure the strength and reliability of which is thus proven with the original material means both witnessing the civilization and a historical event and preservation of the integral part of the history it witnessed and the current environment.

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CHAPTER VII
Additions

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ON SITE AND LABORATORY DETECTION OF THE QUALITY OF MASONRY IN HISTORIC BUILDINGS

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ABSTRACT
Following a long experience on the study of historic masonry characteristics by on site and laboratory tests in seismic areas the authors to are proposing a methodology for the definition of the masonry quality This procedure can be useful for professionals designing appropriate repair and conservation intervention on historic buildings. The methodology is proposed together with some considerations on the use of the test results.

1. INTRODUCTION

The influence of the masonry characteristics on the physical and structural behaviour of load-bearing walls is fundamental. Masonry, especially in historic buildings is often a complex composite material not only due to the variation in origin and properties of its components (brick, stone, mortar), but also to the multiple technique of construction of the wall. In the case of old stone masonry the walls are frequently built with the multiple leaf system, the stones are irregularly cut and the irregularities of the courses are filled with thick mortar joints, while the leaves can be badly connected along the depth of the section. The behaviour of these walls is influencing the safety of the building not only under the vertical actions, but especially under the horizontal actions during seismic events. In the last case the effect of this behaviour can bring to the separation of the leaves and the overturning of the wall under out of plane actions (Figure 1).

Therefore both for repair and prevention purposes, a good knowledge of the masonry characteristics is needed.

Figure 1. Separation of masonry leaves and collapse under out of plane actions

The possibility of gaining this knowledge is given by tests which can be carried out on site and in laboratory; the tests on site should be non destructive or slightly destructive, while the laboratory tests are carried out on specimens sampled directly from the walls; sampling has to respect the existing wall as much as possible, therefore the quantity of the sampled material has to be minimal. This last recommendation obviously excludes sampling of masonry prisms to be tested in laboratory.

Nevertheless the quality of the masonry wall can be detected first of all by knowing its morphology (prospect and section), the physical, chemical and mechanical properties of its components and the properties of the masonry as a composite material. It is easy to describe how to tests the material in laboratory, but the on site testing has to be based on non destructive or minor destructive techniques (NDTs, MDTs). Among the proposed techniques the most effective are without any doubt the sonic and flat jack tests.

A methodology for on site and laboratory investigation is proposed by the authors following an extensive research carried out after the Umbria earthquake in 1997 and experienced in different sites. Some considerations on the evaluation of the results of the proposed testing procedure are also made in the last section.

2. A METHODOLOGY FOR THE DETECTION OF THE QUALITY OF MASONRY

When dealing with the design for conservation, it is important to know the geometrical, morphological, physical and mechanical characteristics of the masonry walls. It is well-known that these characteristics cannot be deduced by the ones of the components, nor using existing standards for new masonry, nor through laboratory tests on materials sampled from the walls. Sampling of masonry specimens is highly destructive and also impossible in the cases as in Fig.1. So only on site testing on masonry are possible and they have to be non destructive. Up to now the only test which can characterize the state of stress and the stress-strain behaviour of a masonry is the flat-jack test (single and double), [1],[2],[3] which is a carried out locally , helped by sonic pulse velocity tests to determine the density and homogeneity of the material [4]. Of coarse perhaps more reliable, but also much more destructive tests are available as shear strength diagonal tests and compression- shear tests carried out on site [5].

Following the proposal made by the 2003 Italian Seismic Code produced by the Civil Protection Department [6] to adopt three different levels of knowledge, the authors have tried to propose a methodology] after the long experience of on site testing they have developed in Umbria after the 1997 earthquake, Liguria and Lombardia after the 2004 earthquake [7].

The Code asks for information on: (i) the evolution of the building through archive documents, (ii) the building geometry and the details on connections between walls and walls and floors and roof, (iii) the crack pattern and damage survey. Furthermore a knowledge is required on the *quality* of the masonry. What the request means about "quality" can be easily clarified. The knowledge should be extended to: (i) the masonry morphology, (ii) the technique of construction (single, multi-leaf), (iii) the component properties, (iv) the mechanical properties of the masonry under horizontal and vertical actions.

The requested knowledge can be reached first of all by surveying the masonry not only superficially through the prospect, but by "looking inside" in order to detect how the masonry section is made. The section geometry is a parameter for the structural analysis and it is also important for the choice of the type of intervention when necessary.

For the choice of an appropriate analytical model the constitutive laws of the materials are needed; therefore the highest number of mechanical parameters is known the most reliable can be the chosen model.

Even if very few parameters can be driven from on site and laboratory tests, nevertheless the experience of the authors has shown that two type of tests can be useful on site: the sonic pulse velocity test and the single and double flat-jack test [8]. The first one is a qualitative procedure which can be useful, when carried out by *transparency*, to find though the distribution of the calculated velocities (Figure 2) the differences in density across the wall. When low velocities are detected (area A in Figure 2b) the masonry might have voids and defect inside, when velocity high peaks are found there might be a connection between the masonry leaves along the section (area B in Figure 2b). Of coarse the results do not give the morphology of the masonry section, which has to be found in other ways. The test can be useful to locate the position of the flat-jack test. The single flat-jack test allows to calculate the state of stress (Figure 2a) in a compressed masonry (by dead loads) and the double flat-jack test allows to find the stress-strain behaviour of the masonry (Figure 2a).

What is needed more in order to qualify the masonry is the section morphology and the properties of the components which can be found by small dismantling of the section and by sampling from the inside mortar, brick and stones. IF all the mentioned information can be referred to the same area of the masonry, then the quality of it can be completely studied.

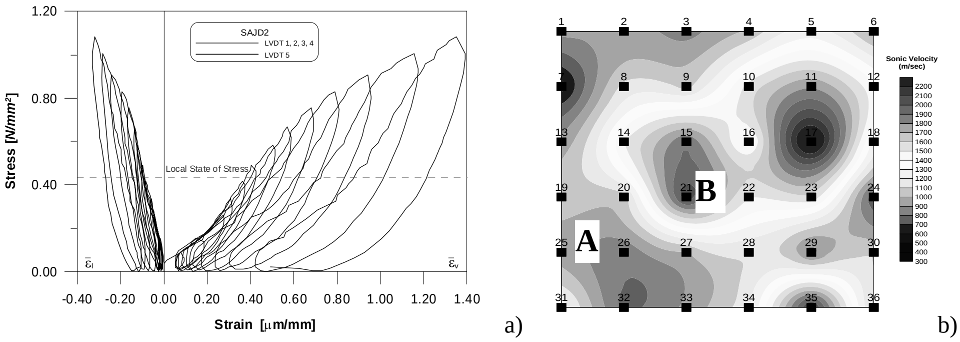

Figure 2. a) Result of flat jack-tests, b) Sonic velocity distribution

It is very important for the chosen area to be representative of the masonry under investigation; the problem on how to find a representative area arises in the case of ancient or old buildings especially in seismic areas which where several times modified or partially reconstructed in the past and in which different types of masonries can be found even in the same wall.

2.1. Description of the methodology

The proposed investigation is carried out according to the following steps: (i) choice of the strategic points on all the types of loadbearing masonries in the building (e.g. choice of a point for each masonry texture observed through the prospect, at the lowest most loaded part of the wall), (ii) survey of the masonry texture of the chosen area in prospect, (iii) sonic pulse velocity test by transparency on a grid of 1mx1m including the area chosen for the flat-jack test; the velocity peaks as said above will indicate a higher density of the material, perhaps the presence of a continue stone or a course of stones crossing the whole section, (iv) single flat-jack test to define the state of stress of the masontry in the chosen area, (v) double flat-jack test with the collection of data, appropriate drawing of the stress-strain curve and calculation of Young modulus and Poisson’s ratio; on the diagram also the calculated value of the stress (by single flat-jack) should be reported in order to see the residual load-carrying capacity of the masonry in the elastic state (Figure 2). The sonic velocity distribution should also be represented (Figure 3a,b), (vi) small dismantling of one or two stones through the section up to ¾ of its thickness, possibly made in correspondance of the sonic velocity peak (Figure 4), (vii) graphic representation of the prospect and section of the wall, (viii) sampling opf a stone and of mortar from the internal part of the masonry in order to be sure that they are the original ones, (ix) chemical, petrographic analyses on mortars, physical and mechanical tests on mortars and stones in order to define their composition and origin in view of a future intervention, (x) repair of the small damge caused to the masonry by sampling.

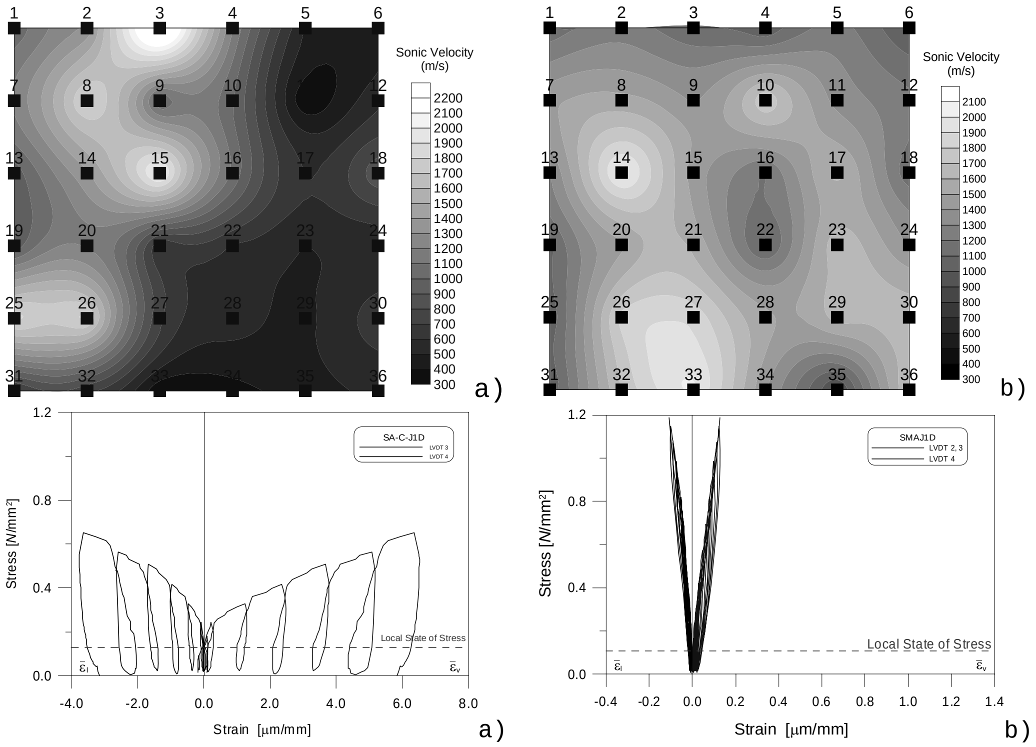

Figure 3 a,b, Results of sonic and flat-jack tests on the masonry of two churches in Lombardia [10].

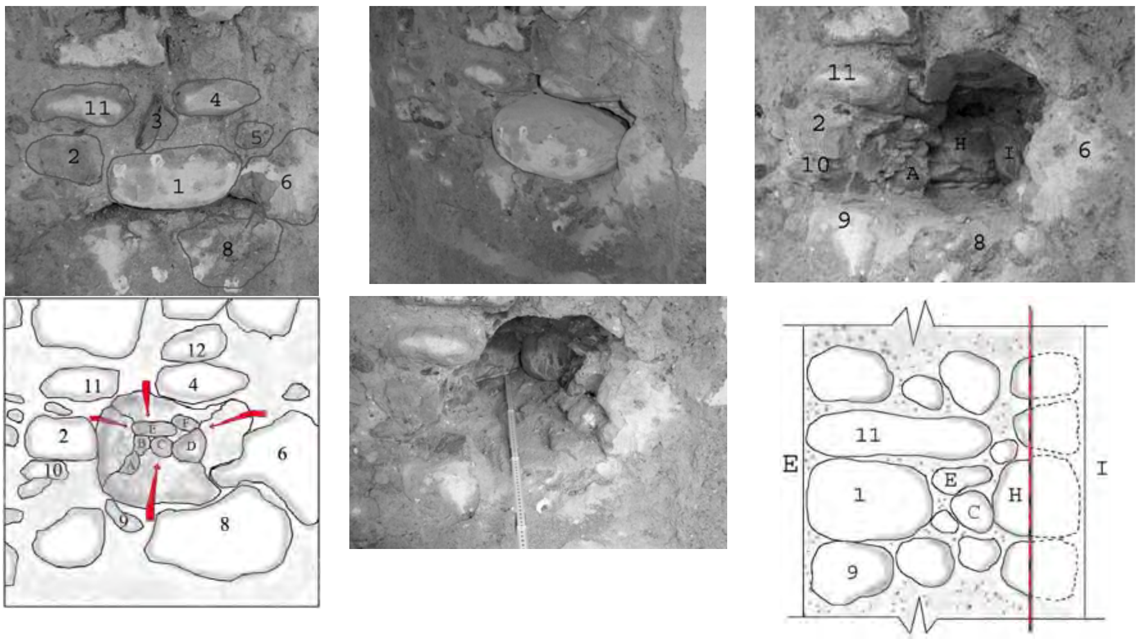

Figure 4. Study of the morphology of the masonry section. Drawing of the masonry texture in the section

It is possible to see from Figure 3 that the highest sonic velocities (Figure 3b) also correspond the the higest elastic modulus (Figure 3b); hence their is a way to define the best quality of the masonry that one which has high sonic velocity and high modulus of elasticity. The description of the masonry quality is completed by the section morphology (Figure 4) and of coarse also by the results of the laboratory investigation on mortars and stones.

3. SOME REMARKS ON THE APPLICATION OF THE METHODOLOGY

Even if ASTM proposes a methodology in the elastic modulus computation, conceptual uncertainties concern the detection of the linear behaviour of the masonry. In fact, due to the masonry peculiar behaviour , the elastic phase is often difficult to estimate. In many cases, locking in the initial phase occurs due to large deformation of the masonry in the first steps of the compression test. Furthermore, since it is impossible on site to reach the ultimate state of the masonry, changes in the curve slope can also be interpreted with great difficulty. On the experience base, the definition of the elastic modulus can be made with different types of computation, but in many cases it cannot be univocally defined. Thus it can create subjective interpretation according to the operator expertise bringing to possible different values.

According to ASTM proposal the value of E can be calculated as:

$$E_t = \frac{\delta\sigma_{mi}}{\delta\varepsilon_{mi}} \qquad \text{tangent modulus} \qquad (1)$$

where $\delta\sigma_{mi}$ and $\delta\varepsilon_{mi}$ are respectively the increment of σ and ε at each step of loading.

$$E_{si} = \frac{\sigma_{mi}}{\varepsilon_{mi}} \qquad \text{secant modulus} \qquad (2)$$

where σ_{mi} and ε_{mi} are respectively the value of stress and strain reached at step *i*.

While the values given by eq. (1) follow the variation of E along the envelope of the loading curve, it is more difficult to calculate E with eq. (2) particularly when a locking phase is present. Frequently the elastic modulus is calculated as secant modulus in the linear part of the σ-ε diagram. In this way the choice of the secant modulus depends much more on the operator decision. Another possibility can be to calculate the secant modulus during the unloading phase, which represent the elastic response of the masonry during unloading. In Figure 5a,b the two ways of calculation of the modulus are represented. Further elaborations are ongoing in order to better understand the differences. The possibility of a quantitative estimation of the physical- mechanic property of masonries, by the use of sonic tests, were proposed within a research carried out in 1993 in laboratory tests and on site on brick masonry of the Veneto Region [9]. Sonic tests in transparency have been carried out on several buildings in order to obtain an average sonic velocity associated to every studied typology. By single and double flat jacks the state of stress and the elastic modulus of each wall were also evaluated. Figure 6a,b shows similar correlation between elastic modulus and the sonic pulse velocity for (a) stonework (regular and irregular) and b) brickwork. It can be clearly remarked that the stonework gives much higher scattering in the results.

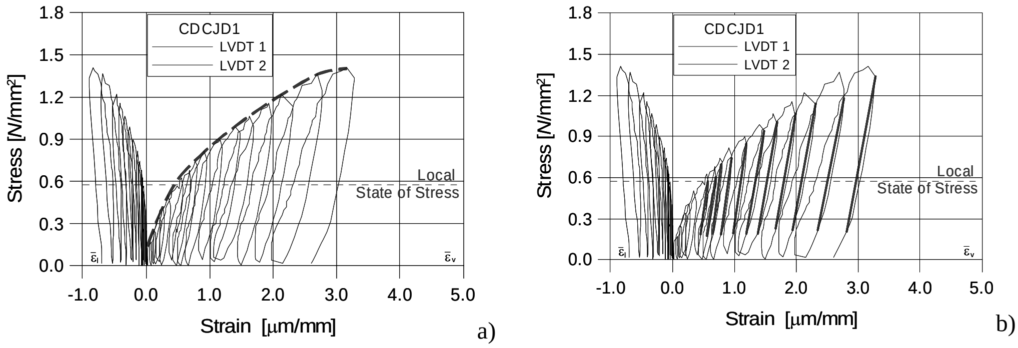

Figure 5. Calculation method of: a) the tangent E modulus according to eq. (1) and b) of the unloading secant E modulus.

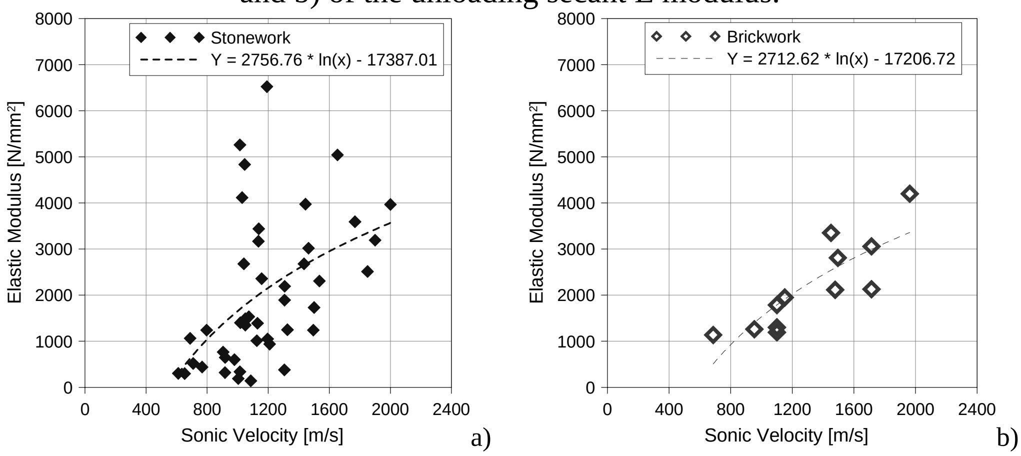

Figure 6. Comparison between elastic modulus obtained from the double flat jack tests and values of pulse sonic velocity measured in transparency in the same area, in case of a) stonework and b) brickwork

4. CONCLUSIONS

The proposed methodology is based on a long term experience of on site and laboratory investigation. The experience with single and double flat jack on masonries, allow to draw the following conclusions: the test with jacks is a valid method for the evaluation of the local conditions of the masonry, since it supplies the value of the state of stress and the stress-strain behaviour. Data can be used with success in the diagnosis of masonry structures and also in the calibration of mathematical models, taking into account that they give only local values. The test can be applied with success to regular brick masonries and stone with thin or thick joints of mortar and on irregular masonries, after a careful analysis of the data. From the elaboration of all the collected data some correlations have been found between the modulus of elasticity and the sonic pulse velocity and the modulus of elasticity can be used to characterize masonry classes.Sonic tests are useful to qualify the masonry density, find the presence of voids and even transversal connections in the section. The other steps of the proposed methodology, survey of prospects and sections, sampling and laboratory tests complete the desired qualification.

5. ACKNOWLEDGEMENT

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SANTA MARIA NOVELLA AND THE DEVELOPMENT OF A FLORENTINE GOTHIC STRUCTURAL SYSTEM

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ABSTRACT

The Dominican Church of St Maria Novella is one of the most outstanding examples of Gothic architecture in Italy. Within the frame of a grant from the Kress Foundation European Preservation Program a research was carried out focused on the nave with the aim of understanding its distinctive design. The investigation carried out on site using non destructive techniques (NDTs) is described together with a first interpretation of the results

1. INTRODUCTION

The first of the great Florentine basilicas to be completely covered with cross vaults, the Dominican church of Santa Maria Novella (1279-1355) has long been recognized as one of the most outstanding examples of Gothic architecture in Italy. It served as a structural model for the nave of the Cathedral of Florence, among others, and continued to be much admired throughout the Renaissance. One of the major factors that accounts for its success as a design is the unified interior space of the nave with its soaring domical cross vaults.

Like their counterparts in other regions of Europe, Italian builders of the Gothic era experimented with new types and methods of construction, such as the domical vaults of Santa Maria Novella raised high on slender shafts, so much more daring than the domical vaulted churches of Romanesque and Gothic Lombardy, and achieved without the aid of the iron tie rods so often seen in Italian Gothic buildings. In so doing, the builders of Santa Maria Novella created that airy interpenetration of space that would come to characterize Tuscan Gothic architecture.

2. RESEARCH OBJECTIVES

The research carried out at Santa Maria Novella focuses primarily on the nave (Figure: 1), with the aim of understanding why and how it came to have this distinctive design. Ultimately, the authors would like to reconstruct the process of design and construction, identifying specific situations where builders made critical decisions regarding design and structure. In so doing, it is possible to gain a better appreciation of the achievements of the builders of Santa Maria Novella as they created what is essentially a Florentine Gothic Structural System. Mathematical models will also be used to better support the results of the investigation.

The particular distinctive features that draw our attention are the use of domical rib vaults, that is, vaults whose crowns change level along the longitudinal and transverse axis, resulting in a sense of a high canopy over each bay, the use of high side aisles with crypto buttressing, concealed above the aisle vaults. A further curiosity is the bay system, in which the first two bays south of the crossing (the nave runs north-south) are rectangular in a proportion of approximately 0.7 length: width, while the remaining four bays are roughly square (see Figure 1). The vault survey was never carried out in a refined way before.

Figure 1- View of the interior of St. Maria Novella

Although the construction of the church is known from documentary evidence to have initiated in 1279, and to have been complete in 1355, very little additional information on the sequence of construction is available. A study of some of the available documents has shown that

- by 1287 the foundations of the entire building were complete and some exterior walls were rising from the ground;

- by 1295 the Feast of Corpus Domini was held in the old church and in that part of the new church that was contiguous, i.e. the crossing;
- by 1301 the eastern half of the old church was destroyed to make way for the first two nave bays; masses are now being said there; this part of the church could be closed off at night with three doors on the front of the screen corresponding to the three naves;
- by 1302 a portal was opened in the third bay of the eastern aisle;
- by 1305 the whole of the church was "roughly sketched out" but much remained to be done inside;
- by 1320 construction of the church was still going on and the *cappella maggiore* was completed;
- by 1325 construction began on the façade wall
- but the entire building was not complete until 1355.

A hypothesis previously elaborated by Smith [1] suggests that the nave was originally proposed to be unvaulted, following Dominican building regulations in force at that time, and was originally intended to consist of seven rectangular bays in the proportion of the first two (0.7:1). At some point during construction, it was decided to vault the nave, and to adopt domical vaulting. At this time, the first two bays were vaulted, and the remaining bays were modified to four approximately square bays, which have been shown [2] to be structurally superior to domical vaults over rectangular bays.

One of the last historical products on the system of vaults in the church is the book written by [3]. The Smith hypothesis was never tested before.

The following methods have been used in the course of the present study.

- Production of Measured Drawings
- Documentation of Cracks and Defects
- Non-destructive evaluation
- Structural Modeling
- Historical Analysis and Interpretation

A particular objective has been to locate evidence to confirm or rule out the working hypothesis outlined in the previous paragraph. This multi-disciplinary investigation has been carried out by means of collaboration between engineers and architectural historians, and collaborations between the Pennsylvania State University and DIS- Politecnico di Milano. DIS is frequently involved in the diagnosis of Italian monuments [4, 5, 6]. The necessity of collaborations between architectural historians and engineers has resulted from the structural character of many of the questions surrounding the construction of the nave and the ability of certain engineering techniques to provide information on construction non-destructively.

3. GEOMETRICAL SURVEY

The geometrical properties which were surveyed in S. Maria Novella (in Figure 2 the plan of the church is reported with the spans interested by the deepest investigation) concerned the following parts and elements: (i) extrados profiles of the two vaults of the central nave named EPV-S2, EPV-S5, (ii) profiles of the pillars and of the intrados of the vaults (IPV-S2 and IPV-S5) of the three naves, (iii) the masonry tympani sustaining the timber roof from the extrados of the load-bearing arches of the central nave, (iv) the horizontal section of the three types of pillars in the church.

The topographic survey of the profile of the pillars and of the intrados of the two vaults called IPV-S2 and IPV-S5 was carried out using a laser integrated theodolite (GEOTOP), in order to check also the pillar verticality. Initially a topographic network (local system) was created, formed by a closed polygonal made of 7 vertexes and constituting the framework of the survey. On it the detailed survey of each element was based, so to minimize the increase of the accidental error inherent to the measurement (Figure: 3).

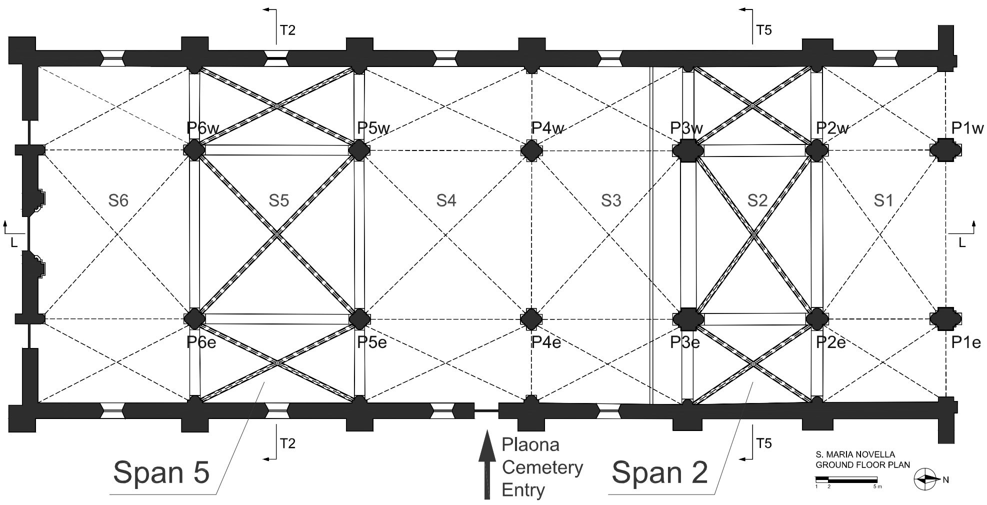

Figure 2. Plan of the church reported with the spans interested by the deepest investigation

The topographic survey of the extrados of the vaults S2 and S5 was also carried out by using the laser integrated theodolite.

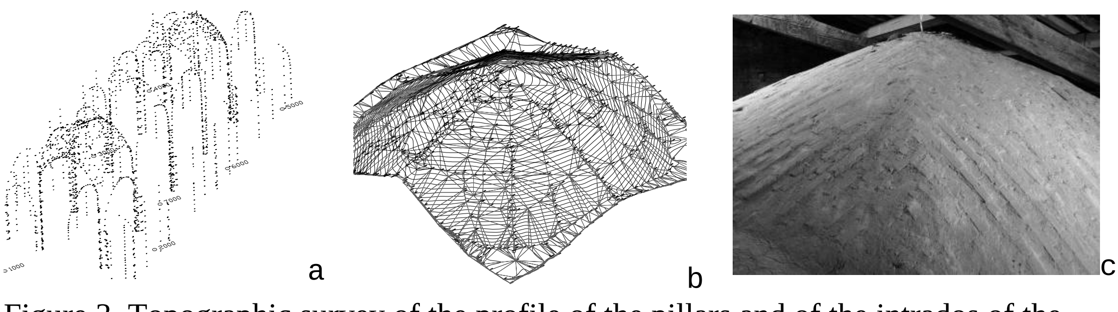

Figure 3. Topographic survey of the profile of the pillars and of the intrados of the vaults (a); 3D reconstruction of the vault extrados (b); view of the details of the extrados of a vault (c)

Also the vault thickness could be measured and was found to be around 30cm. It can be seen from Figure 3 that the geometry of the vaults is extremely complicated. The decisions made during the simplification of the geometry from the super precise total station data to a symmetric geometric model are critical.

4. NON-DESTRUCTIVE EVALUATION

A number of non-destructive methods have been used in an effort to understand the nature of concealed parts of the building fabric. These methods have been applied in complementary fashion, with the findings from each investigation used to substantiate the findings of the other investigation methods.

4.1. Impulse-Echo

The impulse-echo method was used to determine the elastic properties of the material in the vaults and, on an experimental basis, to investigate possible changes in thickness in the vaults. In this method, a sonic wave is generated on one surface of the vault, and an accelerometer is used to record the reflections of this wave on the same surface. Due to the multiple internal reflections of the wave, a profile of the construction of the vault through the thickness can be obtained. By inspection of holes in the vault adjacent to the point of investigation, the thickness of the vault at this location is known to be 32-33 cm. An important peak in the response at 3.4 kHz peak results from reflections at the insider surface of the vault, and corresponds to a wave propagation velocity of 2180 m/s. Using the further relationship that the square of the ratio between the modulus of elasticity and the density of a material is proportional to the propagation of *p*-waves, and the range of density of the bricks of 1700 kg/m^3 to 2000 kg/m^3, it is possible to determine that the modulus of elasticity of the vault material is approximately 9 GPa. Further investigation of the result of multiple investigations over the entire vault show that there are only minimal changes in thickness through the extent of the vault. This is consistent with the findings of the georadar testing of the vault construction and with the geometrical survey.

4.2. Dynamic Testing

The methods of experimental modal analysis have been used to determine the natural frequencies and the mode shapes of the vaults over nave bays 2 and 5. In these studies, an array of accelerometers has been placed on the vaults in 16 locations, and vibrations have been excited in the vaults by means of a controlled force impact hammer. The resulting signals from the accelerometers can be processed to determine the natural frequencies and vibration mode shapes of the vaults. The results are eventually used to compare to the results of analytical models for the purpose of assuring that the analytical model is an accurate representation of the condition of the structure.

4.3. Sonic Pulse Velocity Testing

Sonic pulse velocity tests are carried out to detect the presence of density variation in the walls and hence the presence of different connections in the depth of the wall or of defects and cracks. The sonic pulse velocity tests were carried out in three positions, as shown in Figure 4. Figure 4 also shows the crack pattern surveyed on the extrados of the vaults. If Figure 3 is taken into account the crack pattern can be explained with outward movements of the load bearing walls and pillars due to the thrust of the vaults (no tie rods are present) and the cracks are formed in the weakest points of connection.

Sonic tests applied to three areas were used to achieve qualitative information about the transversal wall between the second and the third span, over the extrados of the vaults of the nave.

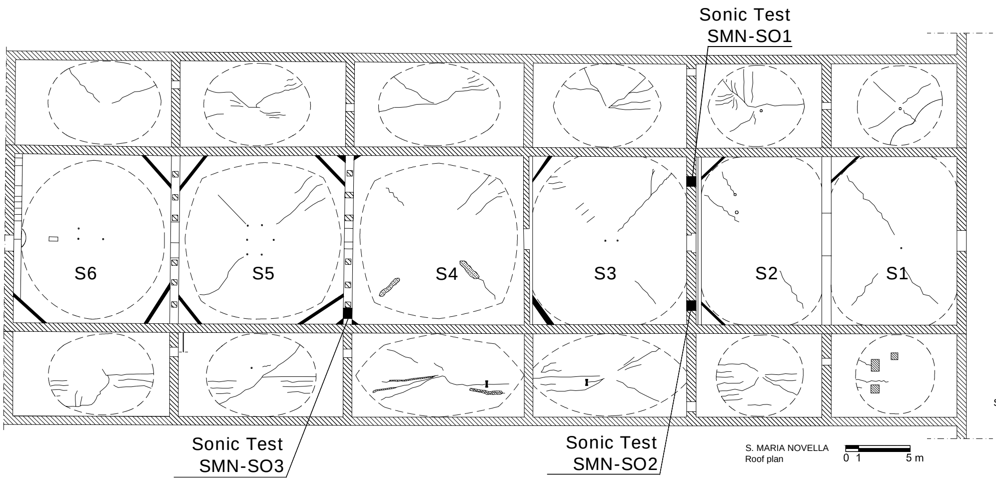

Figure 4. localization of the sonic tests performed in the roof level

Figure 5 shows the location of sonic tests on two grids positioned on the transversal wall. The crack patterns surveyed on this wall and on the other transversal ones, all affected by similar cracks confirms the settlement of the vertical walls and of the pillars due to the vault thrust. The same position of the grids was maintained also in the measurements at other spans.

An average rather uniformly distributed rather low velocity of 1,200m/s was measured while on the grid SMN-SO3 positioned on the transversal wall between the fourth and the fifth span is higher (2,200m/s), showing a probably better construction technology.

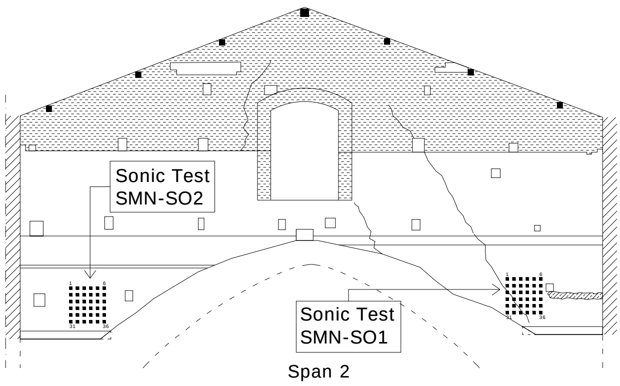

Figure 5. Sonic tests in the transversal wall between span 2 and span 3

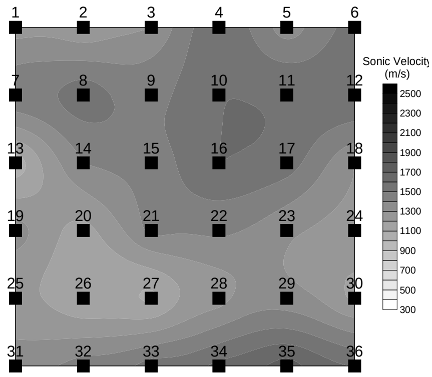

Figure 6. Test results on SMN-SO1 position

Sonic tests were also carried out on some pillars of the central nave made with sandstone. Here an average velocity of 2,800m/s at the base and of 1,500m/s at the level of 1.40m was measured showing a weakest material used in the upper part (Figure: 7).

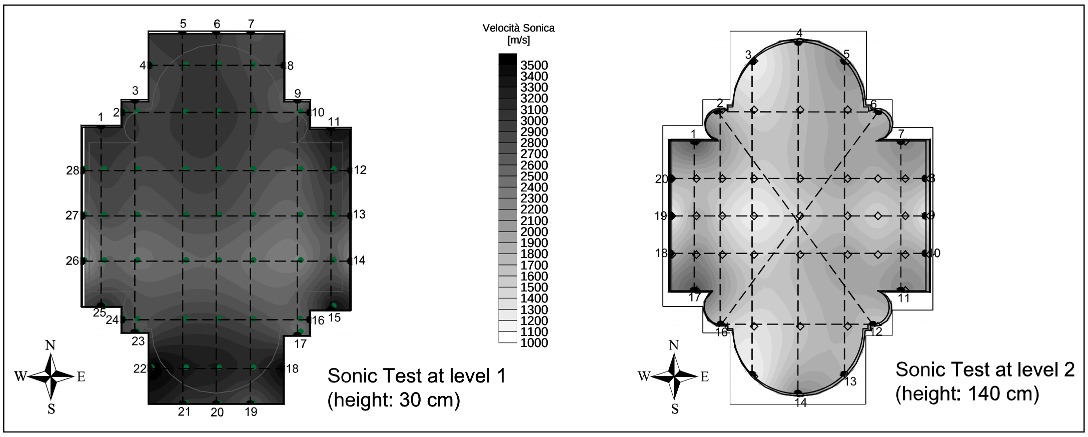

Figure 7. Distribution of the sonic velocity on the basement of pillar P3w

5. STRUCTURAL MODELING

A considerable effort has gone into the development of analytical structural models for the interpretation of the behavior of the vaults and their buttressing system. The modeling procedure has undergone a continuous development since the initial site visit in 2002. The findings of the analytical studies on these models are incorporated in a recently published article [2]. As a result of the comparisons of bay plans and vault geometries, it is evident that a square bay is better suited to a domical vault, while the rectangular bay is better utilized with even-level-crown vaults, and each pair has its own advantages and disadvantages. The transverse load distribution in Bay 2, a rectangular bay, is not as favorable as that of Bay-5, because the former is not perfectly square. Although a detailed stress analysis is not provided and the limited stress results do not perfectly match the cracks observed on site, the Bay-2 vault and Bay-5 walls are found analytically to be more susceptible to cracking, and this general observation is in accord with the condition of the structure.

6. CONCLUSIONS

The multidisciplinary research developed on the vaults of St. Maria Novella was an important help toward the goal of understanding why and how its gothic system has such a distinctive design. Furthermore it gave the opportunity of increasing the knowledge on the process of design and construction, identifying specific situations where builders made critical decisions regarding design and structure and also allowed to surveyed the present damage of the vault system.

ACKNOWLEDGEMENTS

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EVALUATION OF THE SEISMIC VULNERABILITY OF THE SYRACUSE CATHEDRAL: INVESTIGATION AND MODELLING

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ABSTRACT

The authors have intensively studied the Syracuse Cathedral in order to evaluate the structural state of preservation of the building. The Cathedral of Syracuse was built in different phases on an ancient Greek temple from the 5th cent b.C. and modified along the centuries. The pillars of the central nave, obtained by cutting the walls of the temple cell, show a complex situation of damage and repairs. A program of investigation has been recently developed with the objective of studying the vulnerability of the building to seismic events through analytical models as well as of designing eventual provisional measures for its safety.

1. INTRODUCTION

The Cathedral of Syracuse is the result of the transformation of an ancient Greek temple from the 5th cent b.C. with modifications that have also been consequence of the damages caused by earthquakes (Figure: 1).

Peculiarly, the pillars of the central nave, obtained by cutting the walls of the temple cell, show a complex situation of damage and repairs (Figure: 2). The crack patterns, with mainly vertical cracks at the lowest part and on the corners, could represent a situation of progressive damage and need a careful investigation in order to understand the causes. A program of investigation, including sonic, radar, ultrasonic tests, thermovision, structural modelling and crack monitoring, has been recently developed with the objective of studying the vulnerability of the building to seismic events through analytical models as well as to design eventual provisional measures for its safety and long term preservation and restoration actions [1].

2. THE SYRACUSE CATHEDRAL

The Greek temple of Athena in Syracuse, built the 5th century b.C., was transformed into a Christian Church in the 6th century A.D., and successively

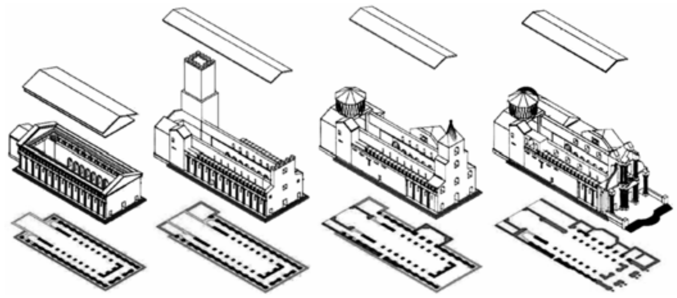


Figure 1. Cathedral evolution before the earthquake of 1169, 1542, 1693, 1800.

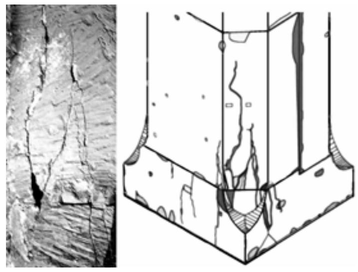


Figure 2. Detail of the cracks pattern of the pillar 26

became the Cathedral of the city; the building was frequently modified along the centuries until the present configuration [5].

Several styles and structural details belonging to the different times can be recognised: (i) in the external walls the ancient Greek columns and the filling wall between them belonging to the Byzantine time, (ii) the baroque façade, (iii) the added apse, (iv) the barrel vaults of the aisles. Furthermore, being Syracuse in a seismic area, the Cathedral was damaged, repaired or partially rebuilt several times.

The most ancient building is the Greek temple while the existing building is a Cathedral with a dome, lateral chapels and a baroque façade [5]. In the VIth century, during the Byzantine period, the temple was transformed into a church with a nave, obtained from the internal cell, and two aisles. The external space between the columns was filled by stonework masonry. The walls of the temple cell were cut, obtaining arcades and pillars (Figure: 3). The works were carried out with precision taking into account the Greek masonry techniques made by large stone blocks and the necessity of the arch rhythm (Figure: 4).

With the Norman invasion, and after some strong earthquakes, the church was modified with a new façade, a new apse, windows in the perimeter walls, a new roof and an increase of the height of the walls.

In the 1542 a strong earthquake stroke Syracuse and caused the collapse of the Norman façade, successively rebuilt, and serious damage to the lateral walls.
Figure: 5a shows a survey of the damage occurred in the 1542 in the north side with a shift of the column drum still visible. The wall was strengthened as shown in Figure: 5b.

The 1693 earthquake destroyed again the façade that was rebuilt as we can see now. There is no information about the effects on the internal walls, even if it is

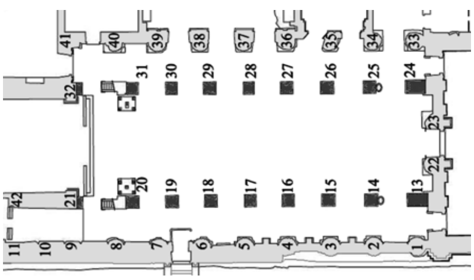


Figure 3. Plan of the Church with the pillars numbering

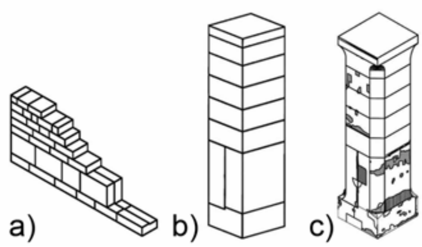
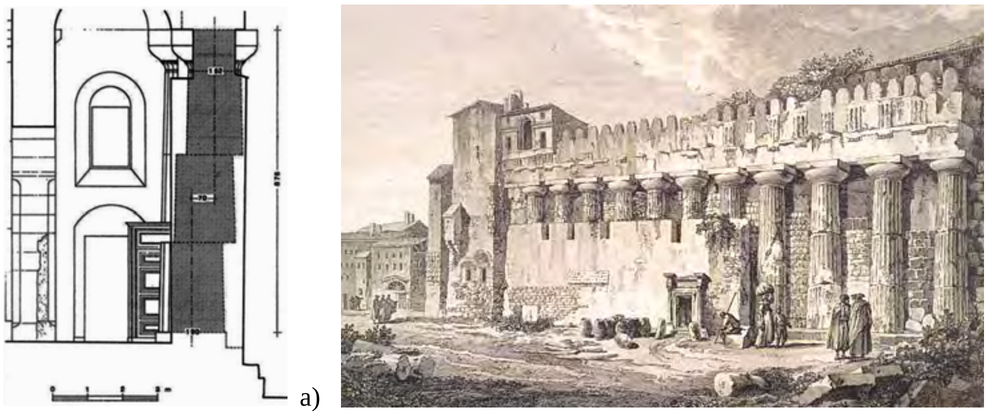


Figure 4. Scheme of the pillar assemblage: (a) original wall, (b) original shape of the pillars and (c) shape after 1924-26 intervention.

reasonable that serious effects could have interested the first pillars and the spans toward the collapsed façade.

At the beginning of the 20th century (1924-26), several interventions were carried out as the removal of the baroque decoration inside the church.



a) b)

Figure 5. Detail of a damaged column (a) and view of the intervention following the earthquake in a picture of the XVIIIth century (b).

3. SURVEY OF THE STATE OF THE PILLARS

The pillars show several repaired areas, replacements, and several serious cracks. As a first step of the investigation program, a detailed survey of the pillars was carried out. This allowed the localization of the most damaged area, suggesting the further control by non destructive testing (NDT) and monitoring [1].

The pillars seem to be suffering progressive damage since long time (Figure: 2), and several of the large stone blocks in the pillar basement show vertical cracks around the corners.

The crack depth, investigated by the ultrasonic tests, demonstrates some serious damage. In some cases, the radar investigations detected the presence, confirmed by direct visual inspection, of unexpected brickwork used, around 1925, to substitute some damaged stones. The out of plumb of the pillars was measured both in the aisles and in the nave. Furthermore, the first results of monitoring show continuous movement of some crack and seem to show the necessity of a repair in the most damaged pillars. Monitoring by acoustic emission was confirmed the presence of internal progressive damage in some pillars.

3.1. Geometry and material survey

Due to the complex situation, the survey of the pillars was carried out with an accurate mapping of the several superficial materials, of the defects, of the cracks and of the morphology (Figure: 6).

The out of plumb of the pillars was measured both in the aisles and in the nave. The horizontal displacement measured on top of each pillar of the right side are substantially higher then the left side ones. The maximum values concern the pillars 25, 26, 27 (Figure: 4) and reach 14 cm. Taking into account the geometry of the church this could be partially explained by the fact that the left aisle of the building is less restrained than the right one (Figure: 7). Nevertheless, the presence of the short span vaults in the left aisle could produce an increase of stiffness for the left side pillars of the nave.

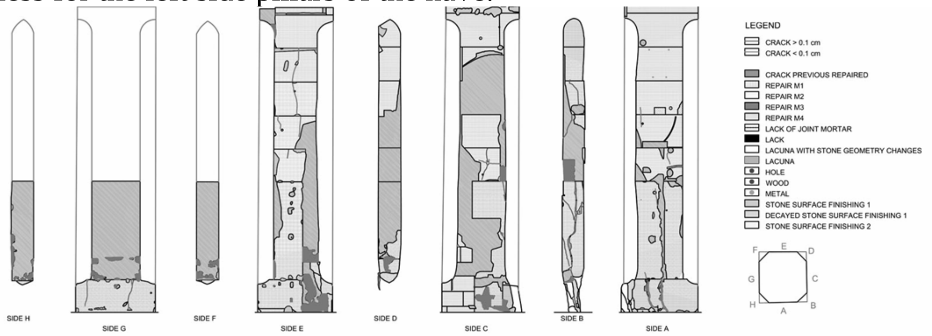


Figure 6. Survey of the pillar 19.

3.2. Crack pattern survey

The crack pattern was accurately documented by pictures (Figure: 2) and reported on the geometrical survey (Figure: 8). A classification of the cracks was made according to their thickness. Peculiarly, the survey has localised the most frequent damage on the base and on the corners of the pillars.

The cracks have frequently a vertical pattern, showing the presence of high compression stresses, caused not only by the dead loads but also by the fatigue effects of successive earthquakes. In some cases, the corners and part of the stone blocks were expelled. The mortar traces in these cases are trials made in the past to locally repair the damage. In the survey, the repaired cracks were enhanced in order to evaluate the evolution of the damage. The differences of the surface appearance were also reported to stress the sequence of the interventions.

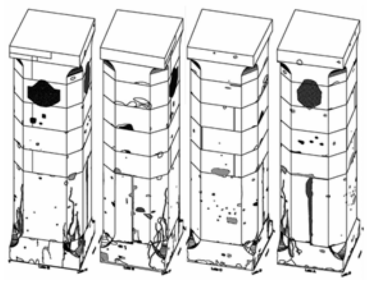


Figure 7. Out of plumb survey   Figure 8. Cracks pattern survey of the pillar 26

3.3. NDE of damages and inhomogeneities

In the case of Syracuse Cathedral, the archive and on-site investigation allowed to understand a number of problems to be solved: (i) the depth of the cracks visible on the surface, (ii) the soundness of the stones, (iii) the depth of the layers of rendering and their bond to the support, (iv) the presence of inclusions, reinforcement, flaws in the stones. All this information could not be collected by a single NDT [1], [2], [9]. Therefore, the following techniques were chosen: (i) sonic and ultrasonic tests to detect the depth of cracks, (ii) thermovision to evaluate the extent of detachment problems affecting the rendering simulating the stone, (iii) georadar as complementary to ultrasonic test and thermovision, to find deep defects and hidden inclusions [1], [9].

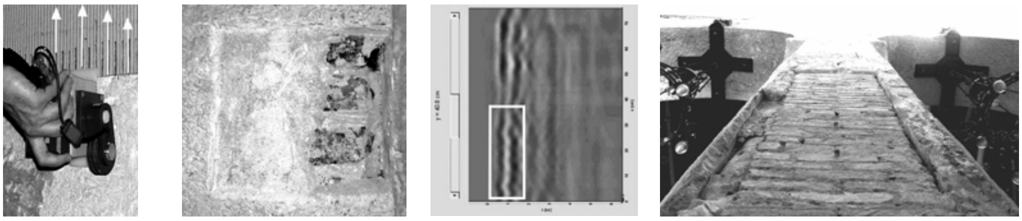
Ultrasonic tests, carried out across the main cracks of all the pillars (Figure: 9a), allow estimating the depth of the cracks (Figure: 9b) [3]. The tests showed that the cracks could have a great depth up to 40 cm.

Thermovision was mainly applied to estimate the detachment of the several types of rendering or re-making of the stone surface applied in the past interventions. In these cases, it was not clear the reason of such intervention and which type of material could be under the covering. In most cases, the covering seems to be detached from the support, revealing the poor compatibility between them. This situation was found in several positions.

Georadar was applied on the Syracuse Cathedral to investigate the masonry morphology beyond the covering and to control the presence of internal defects and cracks of the pillars. The high frequency antenna was also used to estimate the thickness of the rendering applied to reconstruct the external surface of highly damaged pillars. Finally, it was used to find expected and unexpected steel reinforcements applied during past restoration activities. In some cases, the radar investigations detected the presence of regularly spaced diffractions (about every 5-6 cm, Figure: 10a) at a depth of about 3-4 cm, i.e., just behind the plaster. A direct inspection in one of these areas (Figure: 10b) proved that the unexpected brickwork used to substitute some stones during past repairs, probably in the 1920s, produced the diffractions.

a) b) c)

Figure 9. Ultrasonic test procedure (a,b) and result of the tests on the pillar 31 (c)



a) b) c) d)

Figure 10. The radargram shows the presence of regularly spaced diffractions (about every 5-6 cm) at a depth of about 3-4 cm. The inspection proved that these pillars were sometimes repaired with bricks

3.4. Crack monitoring

The experimental investigation showed a very complex situation. Monitoring of cracks was then set up in order to assess possible risks and to understand the mechanical behaviour of the structures before planning an intervention. The first results of monitoring show a continuous movement in the Chapel walls and in pillars 19 and 18.

3.5. Discussion of the results

The research has focalised the problems of the pillars of the Syracuse Cathedral showing that the complementarity of the tests allows a deep knowledge of materials, structures and special features of the masonry.

The pillar 26 is one of the most damaged. The radar investigation shows the presence of metal elements used to join several cracks at the base. Some cracks are extended inside the section.

In the pillar 18 and 19, toward the altar, several damages could be observed. It is important to stress the effects of the 1542 earthquake, which produced a great deformation of the perimeter wall close to the pillars. This could justify the bad state of preservation of these two pillars, characterised by the presence of detached covers and deep cracks on all the prospects. Despite the several interventions the section is reduced. The cracks in these pillars also show a large movement than elsewhere

The pillars 29 and 30 have serious cracks, surface cover and reconstruction in brick and stone, with a low adhesion to the support. The detachments are not only between the covering and the support but also in the stone, due to cracks and to the presence of the expulsive parts. Metal elements, probably ties, demonstrate the tentative of confining the pillar.

4. ANALYTICAL MODEL

A discrete numerical model of the entire construction was implemented with the objective of evaluating the static stress situation under the simple dead load, and also to give an estimation of the effects of an earthquake of relatively small intensity, specific for the site of Syracuse [4].

A three dimensional model was made, by adopting a finite element code, in order to describe in detail both the geometric complexity of the building and the damaged situation of the materials as resulted by the NDT applied on site. The model is very accurate from the geometrical point of view, and was composed by an assemblage of 74,521 brick finite elements, giving a total number of 400,374 degrees of freedom. At this stage, only linear elastic analyses were performed considering: (i) the simple gravity load, (ii) the gravity load plus a constant lateral acceleration corresponding to an earthquake with a period of return of 140 and 475 years, and (iii) a spectral analysis.

The 3-D finite element model was assembled using ABAQUS v6.5 finite element code [6]. In order to have a full control on the distortion of the brick elements, the model was completely implemented “by hand”, avoiding the use of automated mesh generation algorithms. Single parts of the cathedral were modelled separately, and then assembled using a tie constraint. The mechanical characteristics of the materials were assigned on the basis of the results of tests carried out on stone samples taken from temple foundations, and on the results of other studies on monuments built with similar local limestone. Cracks and masonry infills in the pillars were modelled by giving a reduced elastic modulus of single elements according with the results of the diagnostic surveys.

4.1 Eigenvalue analysis and vertical gravity load

The first step in order to understand the global dynamic behaviour of the building consisted in doing the eigenvalue analysis. The first natural mode essentially involves the façade, while the second mode involves the main part of the church, and it is mainly excited by the horizontal accelerations directed along the transversal direction, as shown in Figure: 11, on the right.

In the case of sole gravity load, the mean values for compression stresses were in the range from 0.75 to 1.125 MPa; these values are compatible with the working loads for a good quality limestone, nevertheless there were evident stress concentrations in the pillars that had been weakened by the heterogeneity of the materials.

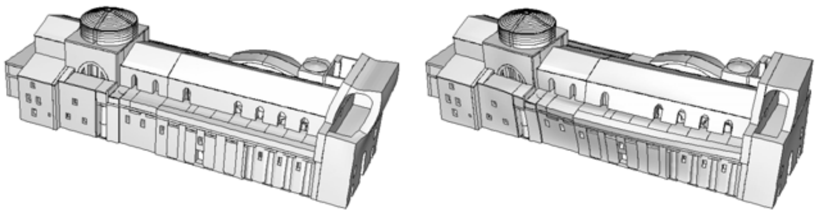


Figure 11. The first two vibrating shapes of the Cathedral

4.2.Static horizontal load

To investigate the global 3-D response to the horizontal actions due to earthquakes, static analyses were performed with a combination of vertical and lateral gravity-type loads. The value of the applied horizontal acceleration was compatible with the earthquakes with a return period of 140 and 475 years for the site of Syracuse [7], [8]. Lateral loads were applied along the longitudinal and transversal axis in both ways. An acceleration of 1.288 m/s^2 was adopted with reference to a seismic event with 30% of probability of occurrence in 50 years (corresponding to a period of return of 140 years). In this case the highest compression stress along the vertical direction reached the value of 1.8 MPa in the nave pillars, as shown in Figure: 12. On the other hand, with an acceleration of 3.066 m/s^2, which corresponds to an earthquake with a 10% of probability of occurrence in 50 years (period of return of 475 years), the maximum compression stress was 2.5 MPa. Moreover, shear stress components reached the value of 0.2 MPa at the base of the northern external wall and in correspondence of the vaults that cover the aisles. The results of the spectral analyses substantially confirmed these data.

The FE model showed that the pillars of the nave are the most critical structural elements of the building. The vertical compression stress, in the case of the sole gravity load, does not reach the strength of a good limestone. In any case the material heterogeneity of the pillars causes some dangerous stress concentrations. When considering a combination of gravity and lateral load, the vertical compression stress reaches values that are close to the strength of the material. Moreover, in this case, also the shear stress components reach dangerous values in the vaults of the aisles.

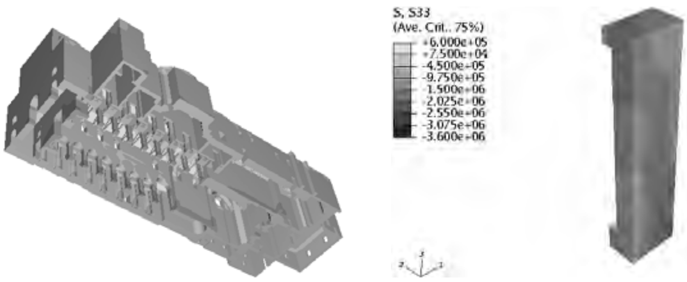


Figure 12. Maps of vertical stress component, σ_{33}, due to vertical gravity load plus a lateral acceleration of 1.288 m/s^2 along transversal direction N→S.

5. CONCLUSIONS

The experimental research has focalised the problems of the pillars of the Syracuse Cathedral allowing a deep knowledge of materials, structures and special features of the masonry.

The surface analysis allowed the localisation of the most damaged area, suggesting the further control. The typical vertical cracks show the presence of high stresses, caused probably both by the earthquakes effects. The crack depths, investigated by the ultrasonic tests, demonstrate some serious damage.

Mapping of the repairs shows the most damaged area, toward the altar, close to the lateral entrance and toward the façade. In many cases, the interventions were not carried out organically and without documentation.

The modelling, that takes into account the results of the experimental results, investigates the linear elastic response in the case of dead load and small earthquakes. Peculiarly, the FE model showed that the pillars of the nave are the most critical structural elements of the building.

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