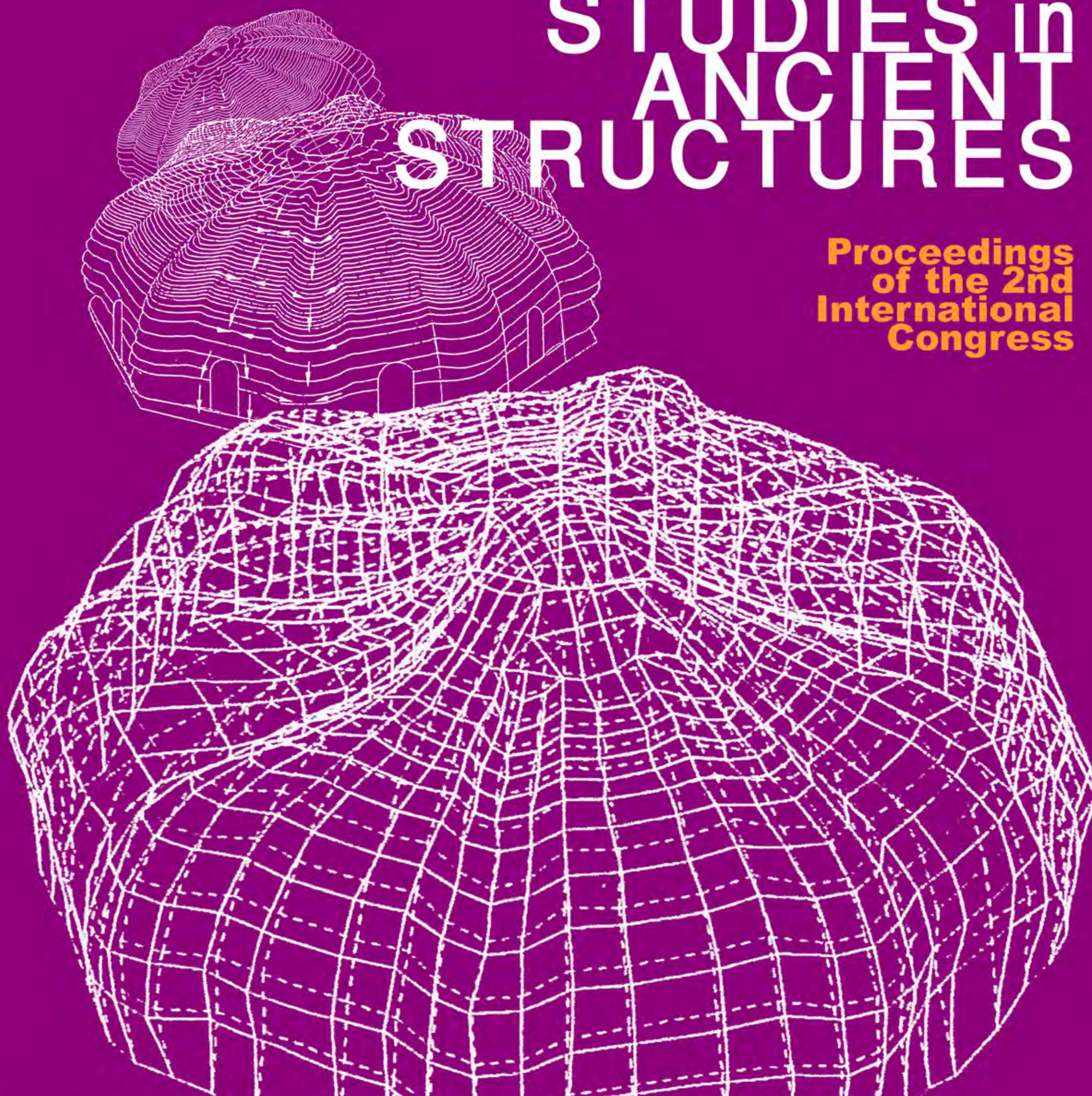
STUDIES in ANCIENT STRUCTURES



Proceedings of the 2nd International Congress

Organized by
YILDIZ
TECHNICAL
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of
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July 9-13, 2001
Istanbul-Turkey

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PREFACE

Ancient Structures referring to the historical buildings are the structures that symbolize the cultural identity and continuity of a land with its architectural, aesthetic, social, political, spiritual and symbolic values. Its age, technological significance with its design, materials and workmanship, association with a prominent designer, being the oldest example of a type or location in a historical setting and representing a period are all notable features to call a structure to be historical. Conservation of historical constructions requires a harmonious work of multidisciplinary team of specialists dealing with history, architecture and different fields of engineering.

The 2nd International Congress on "Studies in Ancient Structures-SAS2001" organized by the Faculty of Architecture of Yıldız Technical University, İstanbul, with the support of ICOMOS-Turkey is held in İstanbul on July 9-13, 2001. İstanbul, the only city in the world built on two continents and served as capital of three empires namely East Roman, Byzantine and Ottoman Empire is one of the most suitable places to communicate the rapid advances made in theoretical approaches, applied aspects and new materials and technologies for studies in preservation of the historical heritage. The Congress aimed at providing an international and interdisciplinary forum for discussion of the studies on ancient structures and to give researchers and practitioners an opportunity to exchange experiences and knowledge on preservation of the historical heritage.

These Proceedings containing papers sent by the specialists of different fields to the SAS2001 Congress is grouped according to their content, rather than according to their presentation, inorder to make it more useful as a reference text. Volume 1 contains Chapters on: Historical and Architectural Aspects of Ancient Structures and Historical Sites; Documentation of Ancient Structures Historical Environment; and Structural Concepts and Analysis of Historical Structures and Sites; Volume II contains Chapters on Experimental Methods and Test Results in Building Materials of Ancient Structures and Historical Sites; Restoration and Preservation Techniques in Ancient Structures and Historical Sites; and Environmental Aspects and Future of Historical Structures and Sites. The author index at the end of each Volume covers all papers in both Volumes.

Each of the papers included in these Proceedings was selected from a much larger group of submittals. Selection was made by the members of the Scientific Committee listed in both Volumes. Deep gratitudes to them for their effort during this hard work. Also many thanks to Prof. T.P.Tassios, Prof. M.Kawaguchi, Prof. S.Kelly, Prof. D.Kuban, Prof. S.Akman and Prof. Z.Ahunbay for their significant contribution as keynote speakers to the success of the Congress. Some of these speaches are also included in the Proceedings.

The Congress is supported by many organizations. Gratefull thanks to our University and Faculty authorities, to UNESCO Cultural Division, TUBİTAK- The Scientific Technical Research Council of Turkey, and other Institutions for sponsoring the Congress. Special thanks to Mr. Ersu Pekin and Mr. Çağatay Bilsel for preparing the illustrations on the covers of the two Volumes and CD, and Arch. Dilek Ekşi, Arch. B.Selcan Yalçın and Eng. Sevcan Yurtsever for their help in preparing the Proceedings and Arch.Olcay Çetiner for preparing the Proceedings on CD. Many thanks to Eng.Timurhan Timur and Eng.S.Emre Pusat of the Organizing Committee for their high performance in every step of the organization. Finally warm thanks to all the authors who undertook the effort of preparing their contribution. It is hoped that these contributions may be useful for professionals engaged in the problems of preservation and for those who have interest in the Studies on Ancient Structures.

Dr. Görün Arun
Chairman of the
Organizing Committee
June, 2001

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CHAPTER I

HISTORICAL and ARCHITECTURAL ASPECTS
of
ANCIENT STRUCTURES
and
HISTORICAL SITES

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INTERDISCIPLINARITY IN STUDYING ANCIENT STRUCTURES

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ABSTRACT

The lecture refers to some complementary studies needed for a more complete and pragmatic structural evaluation and redesign of monuments : (i) Ancient literature review may reveal useful particularities during construction, as well as subsequent damages and repairwork of the ancient structure. (ii) “Leptoscopic” investigations (i.e. detailed visual and instrumental imaging of the structure) offer valuable data, indispensable for a valid mathematical modeling. (iii) “Values” of an ancient structure, other than Safety alone, (i.e. architectural integrity, reversibility and durability of strengthening measures, etc) should be appropriately respected, although they may be contradictory to safety; interdisciplinary optimisation is sought.

1. INTRODUCTION

In addition to specifically structural analysis, studies of several kinds may be useful in understanding an ancient structure and in better decision – making regarding its maintenance, repair or strengthening. As a matter of fact, Structural Engineers may tend to make a seclusion of the concept of “bearing capacity”, without an understanding of the broader historic, architectural and social environment in which the monument was born and used. This may be the case both regarding structural assessment and structural intervention (repair or strengthening).

a) In the first stage **(assessment)**, we tend to be influenced by the philosophy of new structures made of continuous/ one-phase/ quasi-elastic materials:

- In principle, a rational conceptual design is expected to govern an ancient structure (the same way as this might be the case if it were to be designed now). In reality, however, such structures were conceived under possibly different, mostly empirical, state of knowledge.

- On the other hand, we tend to neglect possible discontinuities and gross errors, initially incorporated, as well as accidental events and interventions which have taken place during the long life of the monument.

By way of consequence, in carrying out our assessment studies, we occasionally satisfy ourselves with only i) architectural drawings (much detailed as they may be), ii) descriptions of apparent damages and iii) some "representative" values of the strength of masonry, whatever this may mean. It is time however that, nowadays, additional in-situ investigations are carried out (essentially geotechnical and small scale non-destructive tests) although this is not a rule. Yet, when we come to grips with the structural behaviour of an ancient structure, we often feel powerless; and the best reaction can not be just glamorous finite elements packages…

In this lecture, I will recall some **complementary** sources of valuable information, i.e. ancient literature and "leptoscopic" investigations (giving the emphasis to discontinuities of materials rather than to an abstract continuum.) Both are meant to reduce the lacumae of data, and improve our understanding.

b) On the other hand, when after an appropriate structural assessment, we proceed to the **redesign** of the ancient structure, we occasionally focus so much on the structural safety, that we may not value appropriately the other fundamental performances of a monument (**contradictory** to safety as they seem to be), such as its architectural integrity, as well reversibility and, durability of the added materials. A structural Engineer cannot carry out his duties without a certain knowledge of the significance and the drastic interaction of these non-structural values of a monument with its safety. In this lecture, a formal procedure will be reminded towards a possible optimization of all monumental values.

2. POSSIBLE COMPLEMENTARY STUDIES

Among several sources of information assisting our better understanding of the structural behaviour of monuments, the following additional cases will be discussed here.

2.1. Ancient literature review

In several cases, ancient documents written during or after the construction of the monument, may be a valuable source of structural information regarding structural details or subsequent events (such as incidents, accidents or unusual modifications).

a) The epistyles of the west part of Parthenon ("pronaos") were in a doubtful structural condition; the pathological cause goes back to the 3rd century B.C. A pyromaniac celtic tribe, the Herulians invaded Athens, put fire to Parthenon and disappeared. The timber roof of the temple has concentrated the consequences of the fire to the epistyles; thanks to this historical knowledge, actual restoration work was appropriately guided.

Not without a certain emotion, we had to study in-situ all epistyles by means of pulse-velocity measurements (Fig.1) and other techniques [1].

b) The structural condition of Aya Sophia (Fig.2) in Istanbul is the subject of extensive studies in turkish and international literature. And these studies are frequently making use of information offered by ancient documents. I personally find very interesting the identification of discontinuities and ancient repairs as an extremely important step in analytical modelling of the structure; otherwise we are referring to a "different" structure. To this end, I am simply reminding here some relevant ancient information :

- The incidents to the east main arch and the alarming behaviour of the corresponding piers, during constriction, are first mentioned by Procopius (independently of his, technically wrong, explanations).
- The geometry of the collapse (7 May 558) of the dome, twenty years after temple's dedication, is described by several scholars like Malalás, Theophánēs and Kedrēnós.
- The severe and dangerous cracks of the west arch (and their repairs, 9th century) are described by the emperor Konstantinos Porphyrogennëtos himself.
- The local collapse of the east arch, with part of the dome (19 May 1346), is described by Grëgonás.

Taking into account the related discontinuities, the otherwise visible cracks, as well as the incorporated metal elements (cramps, ties, lead layers on vaults' and arches' springings), contemporary analysts may be more optimistic about the validity of results of their calculations.

c)Another historical damage, the cracks of the cupola of St. Peter's church in Rome, should also be mentioned here briefly. Thanks to the well known detailed work of Giovanni Poleni [3] restorators had a clear view of previous damages, as well as of the expertise of the "three Mathematicians" invited by the Pope Benedetto 14th (as an example of previous analyses, see Fig.3).

d)The cupola of the medieval speyr Dome (Germany) (Fig.4) was damaged during a big fire taken place in the years of 1689. Much later (1893) in a complete monograph on the monument, w. Meyer-Schwartaus published an interesting sketch drafted ten years after the fire. ("Das ist das Profil inwendig von der Cupelwie jetzunt noch stehed"). This finding, together with some (apparently contradictory) repair proposals (1699) (Abb.5), animated lot of later research and interpretations [3], before the final restoration, which took place much later, during the years1970.

e)More frequently, however, history of an ancient building or an urban area may not be described in ancient documents; it is however incorporated in the built environment itself, interwoven with subsequent cultures. The Knowledge of this history is sometimes important for the structural understanding of a monument or of an urban historical building. A couple of this nature are also mentioned here.

- To the left lateral wall of the cathedral of Syracuse, an external strengthening masonry was added (five centuries ago). It is recently discovered that this measure was taken (Fig.6,from [5]) in order to counteract a local shear displacement (about 10%): A doric column (belonging to the temple of Athená) being incorporated in this massive masonry, is now revealed as having been submitted to very large slidings of its drums.
- A typical change in urban development is the gradual relocation of coastal line against sea, due to man-made or torrent infills; the ensuing modification of foundation conditions is apparent in such cases. Such is the emblematic case of Dolma Bachcè in Istanbul, and the transformation of Ortygia from an island (during the Greek times) into the actual peninsula (Fig.7, [6]).

2.2. Leptoscopic investigations

Stone Masonry, the main material of most of our monuments, has escapes long efforts to be understood in a rational way, both as a material and as a structural system. This difficulty may be attributed to our trend to generalizations – or, in other words, to the tendency to underestimate the importance of "details" such as those discussed hereafter:

(i) In elevation, the ratio of the joints' areas and the blocks' areas $\mathbf{(A_j/A_b)_m}$ describes the average normalised "thickness " of the **joints** – an indicator of weakness of masonry ("j" denoting joints, and "b" blocks).

(ii) Still in elevation, possible weak vertical sections should be sought in which the ratio $\mathbf{\Sigma l_j/\Sigma l_b}$ becomes maximum (l_j denoting mortar lengths and l_b block-lenghts); this is a quasi-quantification of a possibly insufficient **stuggering** of block.

(iii) In horizontal cross sections of the wall, it is of fundamental importance to assess the degree of external-to-internal leaf **connection**; unconnected two-leafs or three-leaf masonry, without passing-through blocks, exhibit a particular failure mechanism as this assessment may be, however, its realisation is considerably difficult; local large holes may be needed, unless some more sophisticated non-destructive tests (e.g. Radar) or stereological softwares are employed.

It is easily understood that all this information cannot be substituted by merely a figure expressing the... "compression strength" of masonry, derived from a miraculous empirical formula or from a table.

I have labelled this kind of detailed investigation as a "leptoscopic" (*) one. It is mainly carried out visually, but it tends nowadays to be completely **computerised**, so that a very rapid screening of all the walls of an ancient structure may be possible.

(*) " Leptos" in greek means "Fine"

Yet, considerable research is needed in order to translate this combined information into more concise mechanical characteristics, provided that the strengths of stone blocks, and the mortar [7] are also known.

Leptoscopy is not an essentially "interdisciplinary" procedure; it reflects however a broader attitude and it definitely constitutes a complementary study.

In the same category of studies, other in situ investigations may also be included, such as endoscopy (remote visual examination of some internal details of masonry) and the non-destructive identification of incorporated metal or timber components, as well as hidden discontinuities. The absence of this kind of basic data, cannot be remedied by means of any mathematical analysis of a (non-existing) continuum.

The importance of the aforementioned complementary studies may be illustrated by a couple of examples.

- A three-leaf masonry exhibits a particular behaviour if compared with one-leaf weak masonry of equal compressive strength : Its transversal Poisson-like deformation is so much different (see Fig.9), that creep strength is expected to be very adversely affected}[8].

  Note that under uniform moisture conditions, the leaf boundaries of a multiple boundaries wall may be detected by radar investigations (see i.e. [9]).
- Panthèon of Paris, may considered as one of the first reinforced masonry buildings in structural history (Fig.10). The unexpected side effects of its iron reinforcements (after their extensive corrosion), was a problem which could not be solved without a systematic in situ investigation [10] by means of gamma-radiography (see i.a. [11]), able to detect both the location and the corrosion-level of each iron element.

Once again, a pragmatic analysis of an ancient structure cannot be carried out without complementary studies of this nature; Structural Engineers should be more conscious of this fundamental necessity.

3. OPTIMISATION OF CONTRADICTORY ASPECTS

Occasionally, Structural Engineers tend to consider exclusively one of the many "values" of an ancient structure, i.e. its safety against heavy damage or collapse. Thus, not unfrequently, structural investigations (repairs or strengthenings) may disproportionally affect the architectural integrity or the broader historical value of the monument. Moreover, the structural solution envisaged may not satisfactorily observe the criteria of reversibility and durability established in international regulatory documents. Whenever this is the case, two questions may be raised as discussed hereafter.

3.1. Optimise monumental values (in addition to safety)

Which structural solution should be adopted among several proposals offering the same, conventionally required, level of safety?

To assist decision-making in this respect, a formal technique was proposed [12], [13]. Its essential components are summarised here below.

1st Step

Select a conventionally (**) needed level of design actions (seismic actions included), compatible with the importance of the monument and, above all, with possible occupancy and visitation level.

2nd Step

a)Formulate the absolute minimum requirements for the other performances "P_j":

- Durability D_{req}
- Arch. Integrity I_{req}
- Reversibility Rv_{req}

b)Estimate the "relative importance" of each of the above performances, by means of relevant weighing factors "f_i" such that

$$f_D+f_I+f_{RV} = 1 \tag{1}$$

3rd Step

- Consider several **alternative** investigations (Techniques, Materials, Methods, Extend of intervention)
- Proceed to the preliminary **Designs** of all these "candidate solutions", observing the same basic requirements of resistance R_d against the actions selected in 1st step.
- Estimate the respective global **"costs"**

  C_1,C_2,C_3, ... for each of these "solutions".

  ("Global" means : Costs of
  - credits
  - design
  - education of personnel
  - construction
  - quality assurance
  - social costs during the operation
  - maintenance).

4th Step

- Evaluate the performance levels achieved by each of the above "solutions":

Solution	1	$\rightarrow$	D_1,I_1,RV_1
"	2	$\rightarrow$	D_2,I_2,RV_2
"	3	$\rightarrow$	D_3,I_3,RV_3
"	...	$\rightarrow$	... , ... , ...

- To this end, since quantitative methods are not available, convene a representative Group of Experts. They will assess (be it qualitatively) each of these solutions from their performance point of view (e.g. in terms of classes A, B, C ...).

5th Step

Discard those solutions which do not fulfill the minimal Performance

(**) As imposed by the actual Building Regulations and the State of the art.

requirements you had formulated in Step 2 :

$$\text{Step 4} \rightarrow \quad P < P_{req} \quad \leftarrow \text{Step 2}$$

Calculate the PERFORMANCE MARGINS INDEX of each remaining solution:

$$(PMI)_{\mathbf{i}} = f_D (D_i - D_{req}) + (I_i - I_{req}) + f_{RV} (Rv_i - Rv_{req}) \qquad (2)$$

6th Step

Which solution will be retained now? In Fig.11, three candidate solutions are illustrated in terms of their “total cost (C)” and their “Performance Margin Index (PMI)” available.

Here you may have two alternative decisions : Select method “X” because it offers the highest possible performance level, or select method “Y” because it ensures the maximum benefit versus its cost.

Note

Despite its pseudo-quantitative form, this algorithm is easily applicable in a qualitative way as well. Besides it may help us to organise our thoughts and to minimise arbitrary arguments.

It also makes interdisciplinary action mandatory!

3.2. If needed, could we negotiate the safety level?

Suppose, however, that none of the aforementioned structural solutions was able to observe the really **minimal** requirements regarding architectural integrity and historical authenticity, or those of the reversibility and the durability of the interventions. Whenever this is the case, the following question is often raised : Is it possible to reduce the safety level of the monument, so that a “milder” intervention will be adopted, facilitating the observance of that other (cultural) values of the monument? In an attempt to answer this question, two cases should be examined.

a) When the monument is visited only in its immediate surrounding space (i.e. when its possible heavy damage cannot seriously endanger visitors), the algorithm described in § 3.1 should be **reversed** : Several mild structural solutions are conceived, all observing the minimal performance requirements formulated in step 2 a of the previous paragraph. Each of them (i) lends to the monument a resistance level R_i against the governing actions, whereas at the same time it guarantees that all other performances D_i, I_i And Rv_i of the monument after intervention are by definition slightly higher than the aforementioned minimal values D_{req},I_{req}, and Rv_{req}. Introducing once more the weighing factors of § 3.1, step 2b, we have an interest to select the solution corresponding to the maximum value of the **“MILDNESS” INDEX**

$$\mathbf{M_i = (\underline{R_i}/ R_d) \; [f_D (D_i - D_{req}) + f_I (I_i - I_{req}) + f_{RV} (Rv_i - Rv_{req})]} \qquad (3)$$

Where R_d denotes the conventionally needed resistance-level (§ 3.1, 1st step). The responsibility versus such a higher probability of heavy damage or collapse of the monument is explicitly taken by Society itself (vie its representatives in the group of Experts), since this was the only way to save some of the essential architectural-historical value of the monument. This is a clear case of optimisation strategy.

b) When however the ancient structure is inhabited and or regularly visited, there is no way to accept drastically higher probabilities of failure. Nevertheless, depending on the prevailing rules in the given societal environment, there is a small margin for limited negotiation regarding the minimum required resistance-level : Instead of the conventional resistance-level R_d presented in § 3.1 (1^{st} and 3^{rd} step), a somehow lower value R_o could be accepted, corresponding to a large number of existing inhabited buildings, their design being not based on contemporary Building Regulations; the decision however should be taken by the State – not by the Engineer.

If this is so, now, observing the new minimal resistance requirement, structural solutions for intervention are NOT obliged to strictly observe the "cultural" performance requirements set forth in § 3.1, step 2a, (although this will always remain a desired target). Instead, in this particular case of inhabited monuments, that solution will be selected which leads to the higher NEGOTIATED PERFORMANCE INDEX

$$\mathbf{(NPI) = f_D . D_i + f_I . I_i + f_{Rv} . (Rv)_i} \qquad (4)$$

A final comment on this problem concerns the understandable trend to "animate" monuments, bringing them closer to contemporary life. This however may have two adverse consequences : The first is the additional wear and decay induced to the monument by inhabitants, users or even visitors; in one case, we were obliged to prohibit visitation of the Christian Catacombes of the island of Melos (for several years) because of the oversensitivity of the local tuff against humidity variations and surface wear. The second adverse consequence is that the necessary additional structural safety-level may impose technical measures tending to reduce monumental values such as its historic authenticity.

Consequently, these potential consequences should be clearly taken into account in decision-making regarding the future "use" of an ancient structure.

Definitely, nowadays, we all live in a clearly **interdisciplinary** environment. A bit more complicated as this may seem, it is however much more pleasant and humane, as opposed to some technocratic attitudes of the past!

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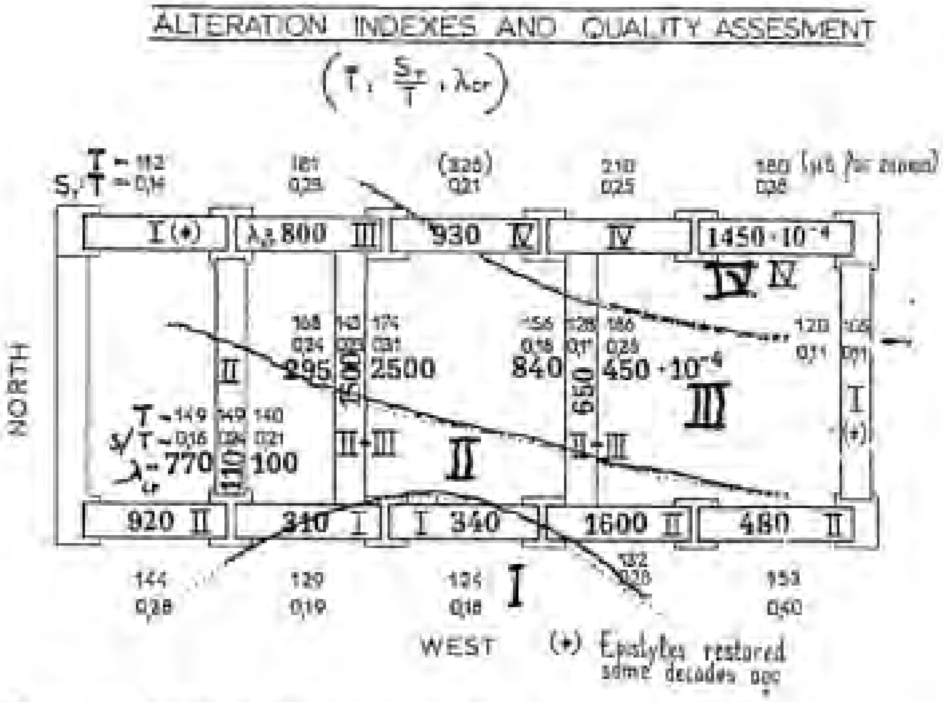

Fig. 1. : Pulse - velocity measurements on the roof elements of pronaos of Parthenon, in order to quantify the structural consequences of the fire put to the temple by the Herulians.

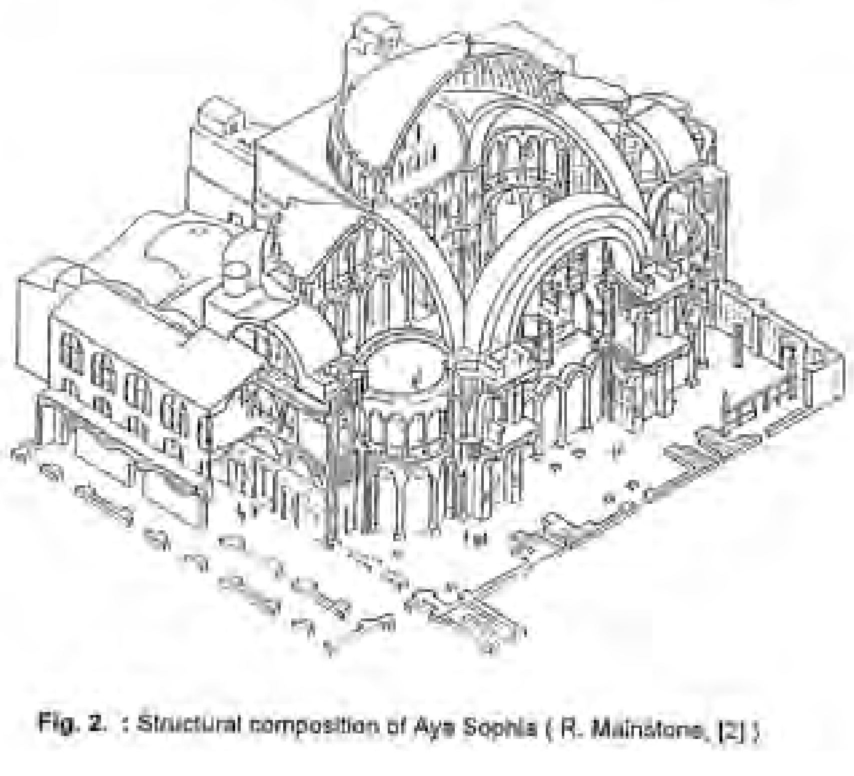

Fig. 2. : Structural composition of Aya Sophia (R. Mainstone, [2])

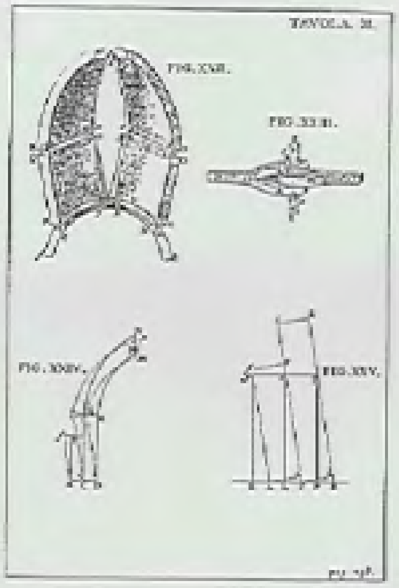

Fig. 3. : Kinematics of cracking of the cupola of St. Peter's dome of Vatican [3]

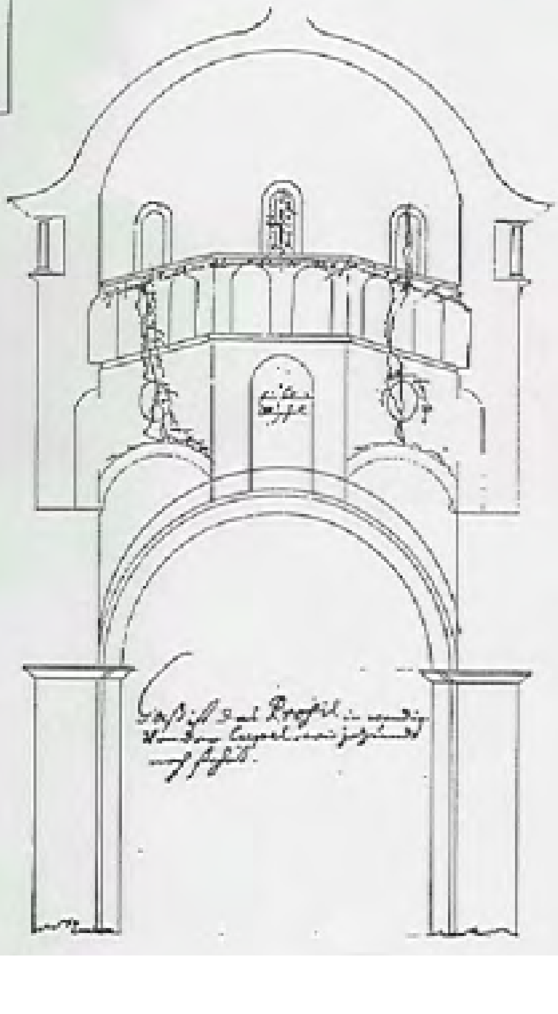

Fig. 4. : A "damage assessment" carried out in 1689 : The Domes of Speyer after a fire [4]

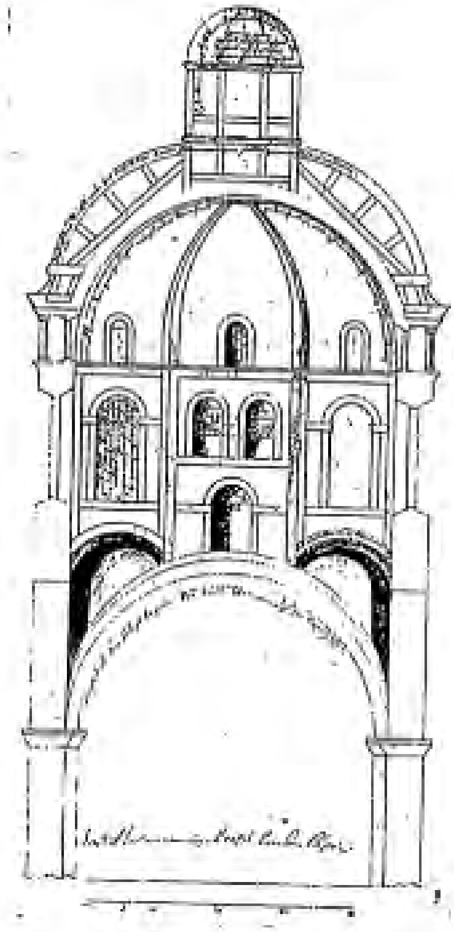

Fig. 5. : An early repair / strengthening drawing of the Dome of Fig. 4, [4], with enlarged windows.

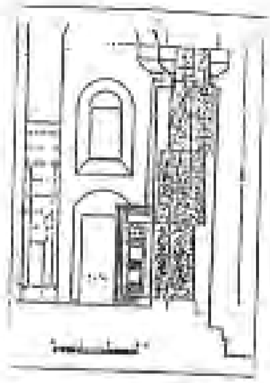

Fig. 6. : A column of the ancient temple of Athena incorporated in the structure of the Cathedral of Syracuse, (Sicily), had suffered severe shear displacements of its drums during the 1542 earthquake; it was subsequently hidden in an external added strengthening wall [5]

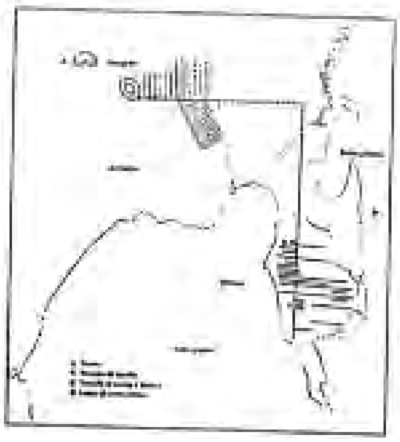

Fig. 7. : Man-made earthfills, during the greek times, transformed the island of Ortygia (Syracuse) into a peninsula; foundation conditions in these areas are different [6]

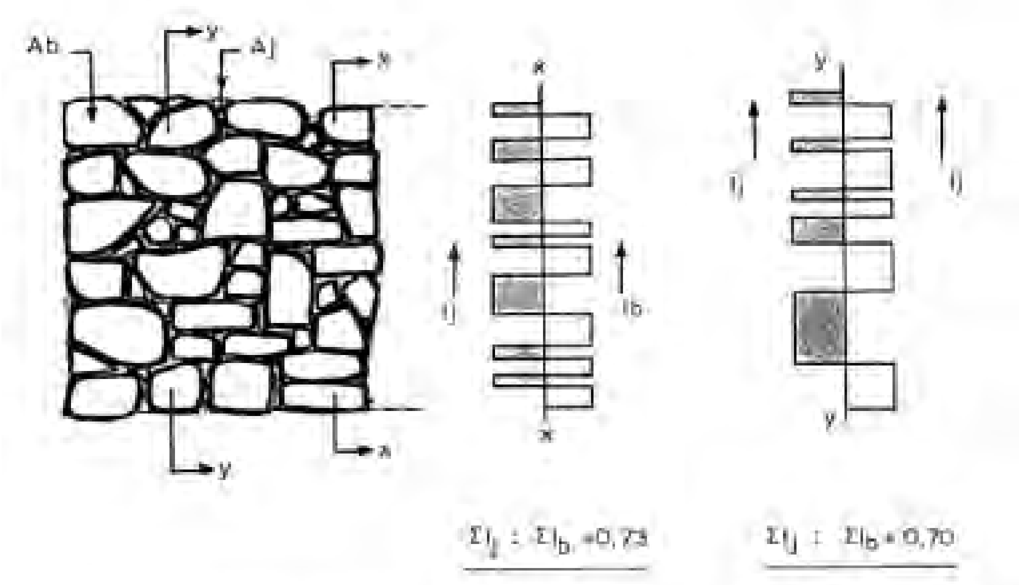

Fig. 8. : "Leptoscopic" view of masonry wall in elevation : (i) Evaluate the ratio of joints - to - blocks areas. $A_j : A_b$, (ii) Seek for several weak-staggering vertical sections, where $\Sigma l_j : \Sigma l_b$ becomes maximum.

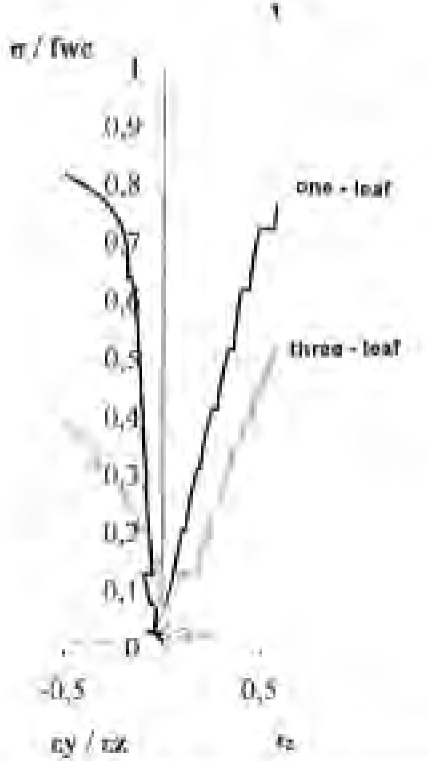

Fig. 9. : Transversal (Poisson-like) deformations of multiple-leaf walls are a basic structural characteristic; creep-strength is better evaluated by means of such deformations.

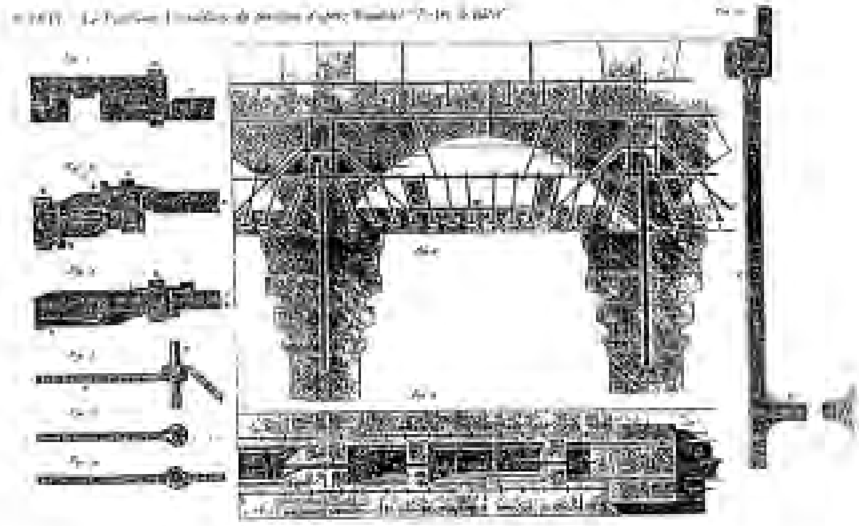

Fig. 10.: Hidden iron reinforcements in traditional masonry of Pantheon, Paris.

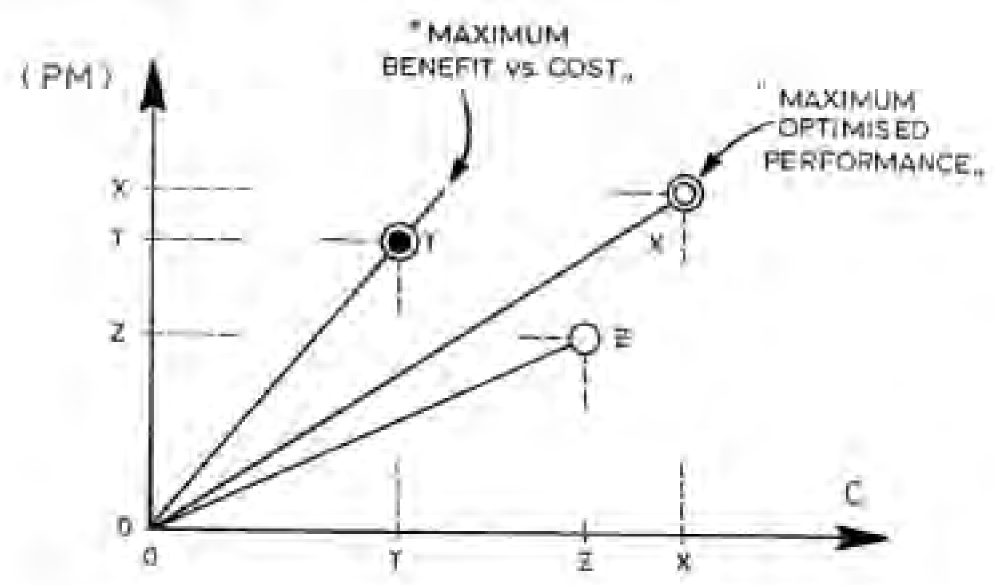

Fig. 11.: A graphical illustration of the "efficiency" of three candidate intervention proposals X, Y, Z

INTERPRETING THE CONSTRUCTION HISTORY OF THE ZEYREK CAMİİ IN İSTANBUL (MONASTERY OF CHRIST PANTOKRATOR)

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Figure1. Zeyrek Camii seen from the east

ABSTRACT

The complex building now know as Zeyrek Camii in İstanbul was originally constructed between 1118 and 1136 as the central feature of the Monastery of Christ Pantokrator, the major religious foundation of John II Komnenos and his wife Eirene [1]. Three churches were built side by side in rapid succession. The south church, dedicated to Christ, was the *katholikon* (main church) of the monastery; the north, dedicated to the Virgin, served the lay community, and the middle, dedicated to St. Michael, functioned as the imperial mausoleum. Although normally interpreted as representing three distinct phases of construction, this paper will propose several refinements to the accepted

chronology, based on a recent examination of the building. The south church was brought to completion before the other two churches were envisioned, but the later two construction "phases" actually may represent a continuous period of building activity, replete with modifications, enlargements, and alterations that were effected only when the construction was well under way. That is to say, rather than representing separate processes, the design and construction occurred simultaneously. This conclusion has important implications for the understanding of Byzantine architectural practices. In this respect, the Pantokrator is *not* an exception in the history of Byzantine architecture, for the picture that emerges accords with archaeological evidence from other buildings, as well as with the written accounts of imperial construction projects from the same period.

2. INTRODUCTION

The church complex of the Pantokrator was the result of growth by accretion, with its three churches, serving three different functions, built in rapid succession. Begun ca. 1118, when John II Komnenos took the throne, the enormous complex was completed by 1136, when the monastic *typikon* (charter of foundation) was written [2]. The south church, the *katholikon* of the monastery, is of the cross-in-square type, with a dome ca. 7.5 m. in diameter, rising ca. 28 m. off the floor, making it the largest example of the building type, and the tallest of the later Byzantine churches in the capital. In plan, it consisted of a monumental block, measuring approximately one hundred Byzantine feet on each side, with the core of the building enveloped by a broad, two-storied narthex and lateral aisles (assuming there was originally a north aisle symmetrical to the surviving, south aisle).

The unified appearance of the south church contrasts dramatically with the familiar jumble into which the complex very quickly developed. The north church was added shortly after the completion of the south church; it was also of the cross-in-square type but smaller and less lavishly detailed. The two were meant to be distinct elements, connected by a single door, where their narthexes joined. Thus, activity in the north church, which was open to the outside community and officiated by a lay clergy, would not have disturbed the worship of the monks, who were isolated within the *katholikon*. As construction progressed, however, it was decided to add a third church, sandwiched between the two, necessitating the removal of the north aisle of the south church. Irregular in plan, the central church is covered by twin domes, with the imperial burials clustered at the west end. An outer narthex and courtyard were added to the south church in this final period of expansion.

In its final form, the significance of the Pantokrator was expressed by its complexity. Separate functional spaces are clearly distinguished on the exterior, identified by their distinctive apses and domes, which were prominent features on the urban skyline. The *katholikon* by itself must have had a monumental and unified appearance, with building elements forming a pyramidal massing around the tall, centrally positioned dome. But as the Pantokrator grew, complexity replaced monumentality as the primary visual expression, and the transformation marks an important shift in Byzantine architectural aesthetics [3].

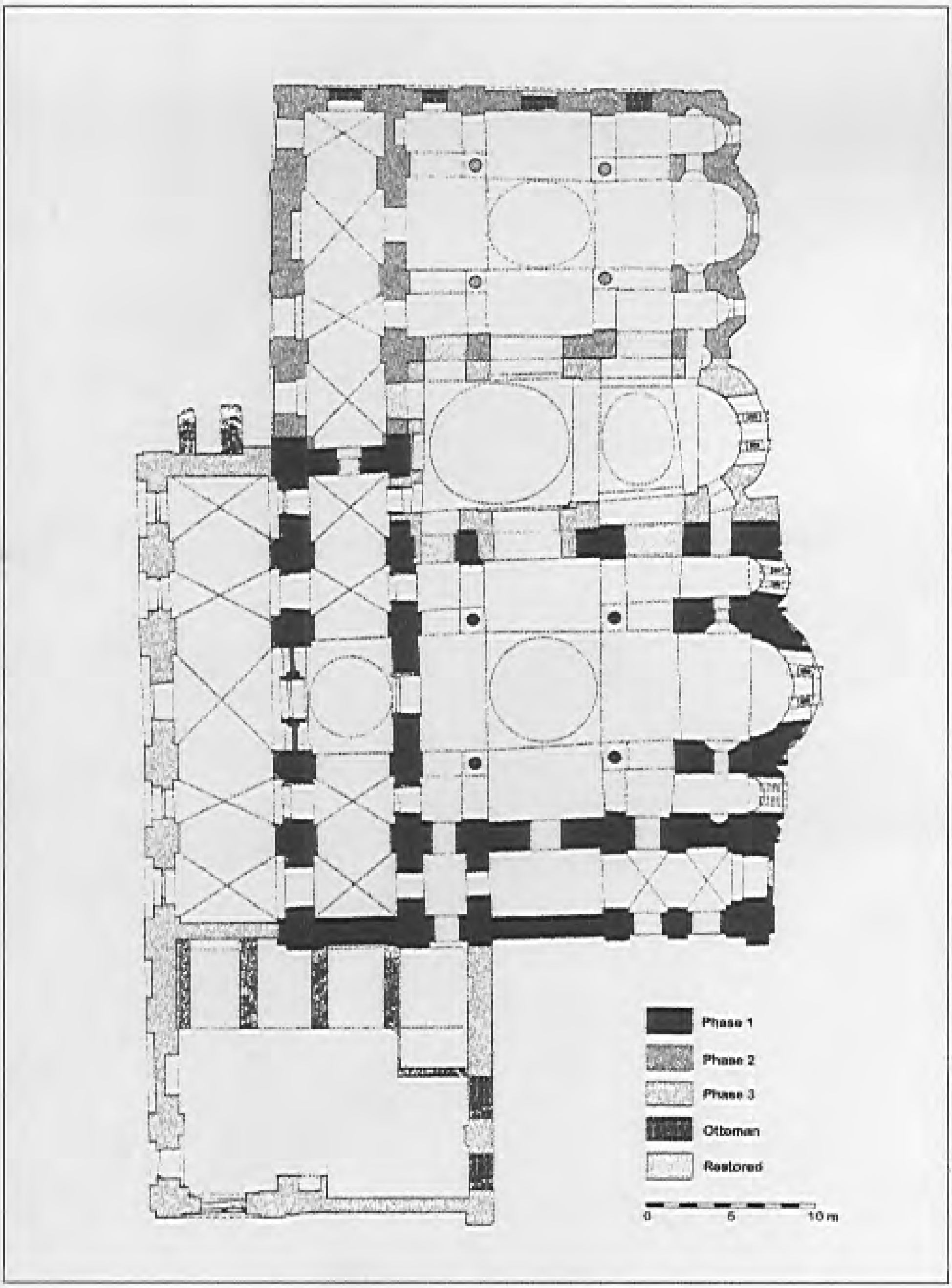

Figure 2. Plan showing phases of construction

2. RELATIVE CHRONOLOGY

The long-accepted chronology of construction, proposed by Van Millingen in 1912, was corrected by Megaw in 1963, following a limited archaeological investigation in the south church [4]. Reversing Van Millingen's interpretation, Megaw determined that the south church had been constructed first, followed by the north and middle churches in successive phases, with the exonarthex and south courtyard as parts of the final period of construction. More recently, in collaboration with Profs. Metin and Zeynep Ahunbay, I undertook a new study and restoration of the Zeyrek Camii [5]. Observations of the masonry at the level of the roof and attic can now add some nuances to the standard three-phase chronology.

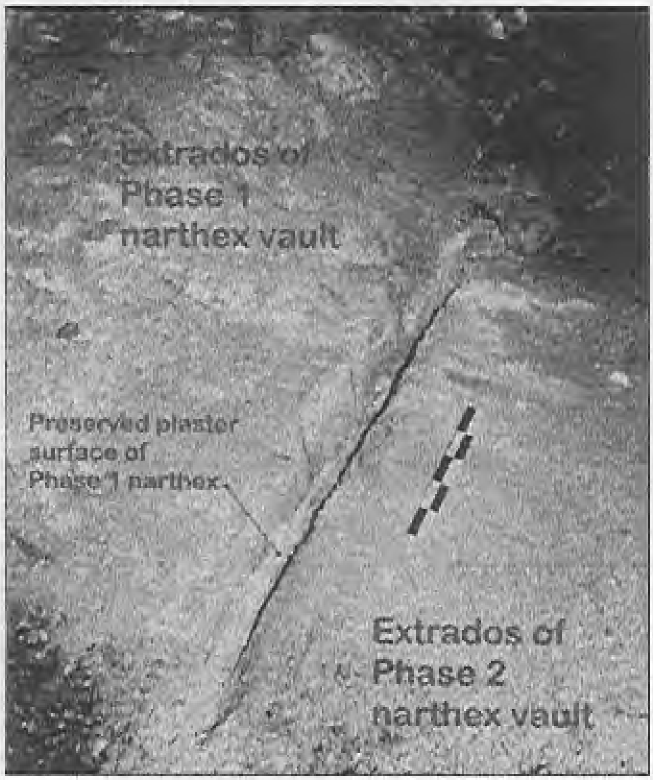

Figure 3. Extrados of the narthex vaults, showing the connection between the south and north churches

It is clear that the south church was completed and plastered on the exterior before the second phase was begun. Both later periods of addition abut the pink plaster surfaces of the first phase, which were left intact. Evidence of the external plaster is visible where the narthex galleries of the south and north churches join—that is, between Phase I and Phase II, and where the dome of the Phase III middle church abuts the Phase I south narthex gallery.

The three-phased chronology may be nuanced with the evidence of numerous sub-phases or changes in design. For example, the unique, twin-domed design of the middle church undoubtedly related to its double function, for it was divided between a liturgical space to the east and the burial area to the west, as the *typikon* suggests [6]. Apparently the western dome was completed first, and then the eastern dome was built against it, with some unfinished surfaces where the two join. The forms of the two domes are distinct: the west dome has a ribbed inner surface, whereas its eastern counterpart is a pumpkin dome. Where they join, windows in the drums interconnect, but the sills and crowns are at different heights. Most likely, the middle church was begun as a single-domed space but was modified during the process of construction. Because the plan was already determined, the east dome had to be constructed above an oblong bay, resulting in its unprecedented oval form.

Figure 4. Domes of the central church, looking south, showing the connection

Another change of design was effected during the construction of the exonarthex. The stepped pilasters and arches of the facade were simplified as the outer narthex was joined to it. The setbacks were cut away to create an angled and slightly concave surface within each arch. Sometime during the construction of the exonarthex, the masons decided to increase the height of its vaulting. On the lateral walls, remnants of lower arches are still evident, framing triple-light windows that were subsequently blocked. The outer narthex was apparently intended to have a lower roof level that would have corresponded with the height of the inner narthex vaults. It is unclear why this particular change was was made, because it has resulted in a space that is lofty but dark, with windows positioned only in the lower walls. Moreover, this change also apparently motivated several other alterations.

Figure 5. View into the exonarthex, looking north

The most important of these was the addition of the gallery dome over the narthex of the south church. The evidence indicates that this was not part of the first phase, but that it was added only after the outer narthex was constructed. Above the present roof, the arch of the central bay rises taller than the others, with its springing at roof level. The pilasters to either side are quite broad, and the profiles of the central arcade are considerably simpler than the others, consisting of two setbacks where the others have four. Within the attic below the present concrete roof, the details of the central arcade are considerably different. There the pilasters have multiple setbacks and are set further apart; between them, a broad arch springs from a lower point. Within the arch are the setbacks for the original window openings [7]. Just above the extrados of the exonarthex vaults, a line of pink plaster is still preserved, which extends into the window reveals. Built against this are masonry additions that correspond to the window mullions visible above the roof. When did the alteration of the central arcade occur? In the attic zone, the mullions stop above a level of unfinished masonry. This indicates that the modification only happened *after* the exonarthex was added—that is, in Phase III, when the unfinished area was already covered by the exonarthex roof.

A final detail is also instructive. In the eastern arch now connecting between the north and middle church, the marble cornice extends uninterruptedly from one building to the next. The apse cornices are set at the same height in both churches, while those in the south church are considerably higher. Although the masonry surfaces in this are are covered with plaster and have not yet been examined, the tentative conclusion we may draw is that construction proceded *continuously* from the north to the middle church. Although in terms of planning it appears as an afterthought, the continuous cornice suggests uninterrupted construction.

Figure 6. View of the roof and domes, looking north

3. INTERPRETING THE CONSTRUCTION HISTORY

Although it was completely irregular in its final form, the building is too important for us to dismiss as simply the unfortunate product of an inept designer. The master mason, a certain Nikephoros, was apparently a man of distinction, who was said to have been the *synergates* (co-worker) of Empress Eirene and "a new Bezalel" [8]. We may puzzle over his design decisions, but he clearly was held in high regard during Byzantine times. Moreover, details of construction, such as the distinctively etched mortarbeds, indicate that the same workshop of masons was responsible for the entire complex.

The numerous design changes and modifications, combined with the complex final form of the building, have important implications for the understanding of Byzantine architecture. The Pantokrator was perhaps the most important imperial foundation of the twelfth century, the result of patronage at the highest social level and the work of a respected master-mason. In fact, the construction history I have outlined in this paper corresponds quite closely to the descriptions by the Byzantine historian Michael Psellos in his *Chronographia* of great imperial building projects of the eleventh century. At the Theotokos Peribleptos, for example, built by Romanos III, Psellos wrote "One on top of another new parts were added, and at the same time another part would be pulled down. Often, too, the work would cease and then suddenly rise up afresh, slightly bigger or with some more elaborate variety" [9] At St. George of the Mangana, according to Psellos, Constantine IX had the design altered and expanded three times during its construction [10].

Although the ironic tone of Psellos' remarks reflects badly on the patrons (as the writer had intended), his descriptions may in fact represent actual architectural practices. Like the Pantokrator, the final forms of many Byzantine buildings are similarly the results of growth and transformation, as occurred at the Kariye Camii and the Fenarı İsa Camii in Istanbul, at the Nea Moni on Chios, at the *katholikon* of Lavra Monastery on Mount Athos, at the Hodegitria in Mistra, and in many other important buildings of the tenth through fourteenth centuries [11].

4. CONCLUSIONS

The evidence from the Pantokrator emphasizes that design and construction were not separate processes in Byzantine architecture, but that buildings were undertaken with a minimum of advanced planning. Details of the elevation could have been finalized only after construction had begun, additions could be introduced, and numerous other design decisions could be reconsidered. We may venture to suggest that new ideas and new building types in Byzantine architecture may have developed in exactly this way—that is, through the modification of existing buildings or through changes effected during the construction process. Finally, we emphasize that the transformation of the Pantokrator through its short construction history marks a significant change in Byzantine architectural aesthetics, with the monumental unity of earlier architectural projects replaced by complexity as the primary visual expression.

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7. Ousterhout, "Contextualizing," fig. 16.
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XERXES' BRIDGES ACROSS THE HELLESPONT STRAIT ACCORDING TO HERODOTUS

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ABSTRACT

According to the author's hypothesis, the bridges that Xerxes built across the Hellespont were floating bridges of the raft type. In this type of bridges cables made a stabilizing element which was to hold rafts in a position, and the ships were used as a protection against the destructive effect of sea waves. However, in all manuals and encyclopaedias the crossing is described as bridges whose supports are made by ships upon which a deck suspended on cables is situated.

1.INTRODUCTION

Herodotus of Helikarnas (485 – 425 B. C.) being quite deservedly labelled as 'Father of History', wrote the work entitled 'A Discourse on History' popularly called 'History' [3]. The work comprised in ten books was highly estimated already by the contemporaries who adjudged Herodotus a huge sum of ten talents as reward. And so, in Book VII, he described the greatest in ancient time campaign of the Persians against the Greeks. The Dardanelles, a strait separating Europe from Asia was of the utmost importance in that historic event.

In Herodotus' days the Dardanelles Strait was called the Hellespont, 'Sea of Helle'. According a Greek legend of Argonauts' expedition the name of Strait comes from the name 'Helle', a girl who fleeing her stepmother on a ram with golden fleece, on her way to Colchis, the country situated at the foot of the West Caucasus Mountains, fell off and drowned in the sea (the name of the country appeared for the first time in Ajschylos' tragedy about Prometheus). It was the golden fleece of Colchis hanging in an oak tree in Ares' holy grove that the Argonauts sailed with Jason to search for (on the groundwork of this legend Euripides wrote his tragedy 'Medea'). Argonauts sailed on the ship Argo (Swift) which had fifty oars. And so, one of the two types of ships which the Persian

King, Xerxes, ordered to use while building the bridge tracks over the Hellespont were the ships with fifty oars.

2. PREMISES OF BUILDING THE BRIDGE TRACK ACROSS THE HELLESPONT

Xerxes became the king of Persia in the year 485 after the death of his father Darius. In the year 484 he conquerred Egypt and decided to undertake a campaign against the Greek States in most cases associated into the Hellenic Union. During the gathering with the Persian seniors he revealed the following aim of his campaign: '*The sun will not get a sight of any land which would abut on ours, but I will combine all of them (...) into the one and only country having marched whole Europe'* [3, VII, Ch. 8]. He intended to do it with the help of his formidable fleet of warships and several thousand in number land army. On the road to rule over the country having Asia and Europe within its border, stood Hellas, and the way from Asia to Europe led through the Hellespont. In order to convey the army across the strait it was necessary to build the bridge track.

3. CONSTRUCTION OF BRIDGE TRACKS DESCRIBED BY HERODOTUS [3, VII, Ch. 33-36]

Translation into Polish – Jerzy Mankowski.
In parenthesis supplement of the original text has been given.

"On the Chersonese, which is by the Hellespont, between the town of Sestus and Madytus there is a rocky coast running down to the sea opposite Abydos (...). To that coast, those who were ordered to do it were throwing a bridge track from Abydos. The Phoenicians – one of flaxen cables, the Egyptians – the second of papyrus cables. From Abydos to the opposite shore it is a distance of seven furlongs. When they were joining (the framework of the structure supporting) the bridge, suddenly a violent storm broke out bursting and smashing into pieces everything.

When Xerxes learnt about that, he was very angry and he commanded to give the Hellespont a flogging of three hundred lashes. I have even heard that he sent people who were to brand the Hellespont with burning-hot iron and to pillory it (to immobilize). He also put those who scourged under an obligation to utter words really savage and furious:

'*Bitter water, our Master inflicts a punishment on you because you did harm to him, although you had not met with any wrong at his hands. What is more, Xerxes the king will get across you whether you want it or not.'*

That was the punishment he assigned to the sea, he also ordered to behead those who had supervised bridging over the Hellespont.

Other master-builders joined (the Hellespont shores) in the following way:

They put fifty-oared ships and triremes together – 360 on the Euxine Sea side and 314 on the other side. They placed them obliquely to the line of the Pontus and parallel with the current of the Hellespont in order to secure the strain of the cables. Having set (the ships), they threw down very long anchors, ones – towards the Euxine Sea because of the winds blowing from that sea, the others towards the West and the Aegean Sea to hold fast against west and south winds. In three places they left a narrow opening for passing between fifty-oared ships and triremes to make it possible for those willing to sail small ships towards the Euxine Sea or from the Euxine Sea to the outside (towards the Aegean Sea) to do it.

After they have done that, they tightened the cables from ashore stretching them tight with wooden hoists. Cables were not taken separately, two flaxen and four papyrus cables were allocated to each (track). They (cables) were thick and of good quality. Those made of flax were said to be heavier. An ell of that cable weighed one talent.

Once (the framework of the structure supporting) the bridge was spread out between the shores, on the tense cables sawn-up logs, equal the width of the bridge, were laid from above, one after the other. Having done that, they joined all (elements) once again.

Then by turns wooden stuff was brought and laid (on the bridge) and soil was put upon it... After the earth had been levelled a stockade was raised on both sides of the bridge so that the animals wouldn't be afraid.

4. PASSING OF THE PERSIAN ARMY ACROSS THE HELLESPONT [3]

It took four years to organize the invading army. As far as manpower of the Persian army is concerned, it has had no equal in history. Ground forces moving on foot had 1 700 000 soldiers, and those riding on horseback had 80 000 (except for camel riders). Naval army amounted to 1207 ships – triremes. Ground and naval army which arrived from Asia comprised 2 300 000 soldiers altogether (besides servants, concubines and eunuchs). When bridge-track building operations were completed, the army, after spending the winter, at the beginning of spring, in the middle of April 480 set off from Sardis to Abydos.

Before crossing the strait, the Persians waiting for the sun to rise, burnt on the platforms perfumes of various kinds as well as strewed their path with myrtle-branches. When the sun rose, Xerxes poured oblation out of a gold bowl into the sea, praying to the sun that nothing would hinder him from conquerring Europe. After the prayer he threw the bowl with a gold mixer-arm and a Persian sword (short and wide) into the waters of the strait.

The army, infantry and cavalry crossed the Hellespont over the bridge located near the Euxine Sea. Munition staff and beasts of burden went over the other bridge, situated on the Aegean Sea side. The army was passing along the track incessantly for seven days and nights.

After having been defeated in the naval battle of Salamis (23rd September 480 B. C.), returning from Europe to Asia, Xerxes and his 60 000 soldiers crossed the Hellespont on ships as the bridge track had already been destroyed [2].

5. THE DESCRIPTION OF BRIDGE TRACK OVER THE HELLESPONT IN SOURCE-BOOKS

In all kinds of encyclopaedias, manuals and articles the bridge track across the strait is presented as two floating bridges [1, 6, 8, 9, 10]. Supports of each bridge are made by ships equipped with a deck suspended on cables [8, 10] or laid on the ships [6, 9]. In the latter instance, cables appear to be a longitudinally stabilizing factor for thus constructed structure.

Nevertheless, it is not evident from Herodotus' text that arranged in lines, crosswise the Strait, ships were used as bridge supports nor that the cables were laid on ships. The fact that such an engineering solution was taken for grated by men of learning, resulted in all probability, from the following grounds:

For one thing, from not too thorough study of Herodotus' description of the passage. By way of example, from the description of constructing the first crossing it appears that it was built with the use of cables. There is no reference to employing ships.

Secondly, in his work Herodotus used the Ionic dialect, which has always been quite difficult for the translator to render.

Thirdly, the description of constructing the bridge track seems to be very laconic and sometimes calls for logical association of facts quoted by Herodotus in various parts of his work. In Chapter 25 he mentions the construction of a bridge track on cables over the River Stymron without giving any information about the use of ships while building it. The above fact might lead one to believe that in cable bridge track building technique used by the Persians there were no ships used as supports.

Fourthly, practice and experience in building pontoon bridges exerted an influence.

Moreover, scholars question the number of ships placed crosswise the Strait – 360 on the Euxine Sea side and 314 on the Aegean Sea side, which was presented by Herodotus. They argue that taking into consideration the size of a ship; it is unfeasible to arrange them in one line in the Strait width [10]. Meanwhile from Herodotus', text it does not follow that the ships were laid in one line on either side.

6. CONSTRUCTION OF THE FIRST BRIDGE TRACK ACROSS THE HELLESPONT ACCORDING TO THE AUTHOR

Building a bridge over a sea strait was a very difficult undertaking and it is not likely that while constructing it, some innovative design solutions could be applied. Most certainly familiar and verified technique of building tracks across water obstacles was used instead.

The passage over the Strait was made by two detached bridges of raft type. They were located in the narrowest part of the Strait amounting to 7 stadia (1243.20 m; in the year 480 the Evvoian-Attic measure system was current in Athens – one Attic foot was equal to 0.296 m, and one a stadium had 600 feet – 177.60 m).

First of all cables were made: the Egyptians prepared papyrus cables and the Phoenicians those of white flax. Both papyrus and flaxen cables were equal in diameter. Next starting from the Asiatic and heading for the European coast, they tried to pull on water across the Strait, wooden clogs which made elements of the bridge supporting structure and on which cables were laid. While they were busy doing that, a storm began, which caused violent roll of the sea, which completely destroyed the floated construction.

Using familiar and verified floating bridge building technique, the constructors did not take into consideration the scope of the design. To build a passage across a one-hundred-meter wide river is one thing and to construct a track over a sea Strait, which is over 1000 meters wide, is another.

Considering different specific weight of water in the Aegean and the Marmara Sea, in the Hellespont there is a dual mainstream – surface and subterranean [1]. The surface stream is very strong and it flows towards the Aegean Sea. In the Strait there also occur whirls, especially by the European shore.

About the current making sailing on the Strait waters very difficult says the legend of ‘Symplegads’ – rocks in the Strait which join each other thus destroying the ships sailing between them. Also Homer, in ‘Iliad’ mentioned ‘the stormy wave of Hellespont’ [4].

7. CONSTRUCTION OF THE SECOND BRIDGE TRACK ACROSS THE HELLESPONT ACCORDING TO THE AUTHOR

The constructors drew conclusions from the failure they experienced during their first attempt to build the passage. This time, constructing the track, as a break-water, they used ships arranged in a line and made to adjoin each other’s side.

Two types of ships were used: fifty-oared ships and triremes. A fifty-oared ship – penteconter (penteconta – fifty) was a ship which could have 25 sailors on either side [7]. In the first decades of the 5^{th} century B. C. Ships of that kind were already out of date and their operational usefulness was of little value (Herodotus does not even give their number in the Persian army). Meanwhile a trireme – triera, was believed to be the best warship in the Mediterranean Sea in the ancient times. It was equipped with two anchors made of metal or stone weighing up to 25 kg each.

While setting the ships, the constructors took into consideration both the direction of the mainstream and the direction of winds blowing in this area. On the Black Sea (Euxine Sea) side the line of ships was arranged in such a way that their centres of gravity laid on the line marking the northern direction, their position being parallel to the mainstream. This kind of arrangement was advantageous as first of all the ship-line surface exposed to winds blowing from the North was the smallest and secondly, it secured permanent tightening of anchor ropes (In Homer's 'Odyssey' [5], it was the Wind Boreas having his seat just there, in the North, who made it impossible for Odysseus to return from Troy to his home Island of Ithaca).

On the Aegean Sea side the row of ships was set in such a way that their centres of gravity marked the line perpendicular to the water current direction. This arrangement was advantageous for the ships because of the winds blowing Southwest and it ensured permanent tightening of anchor ropes.

There is no foundation for questioning the number of ships used, given by Herodotus, as it is not unlikely that in places of more than usually strong water current (on the European side of the Strait), the ships were set in more than one line. Three gaps were left between the ships in order to make it possible for small ships to sail.

Thus arranged ships, functioning as breakwaters ensured in protected water region, conditions similar to those, which can be observed at inland water obstacles. They successfully suppressed sea roll and slowed down the speed of surface water current. Consequently one might say that Xerxes' command to treat the Hellespont as a slave and to put it in double irons was carried into effect.

Building the second bridge track exactly the same construction technique was used as while building the first one, but this time the work was carried on under cover of breakwaters. After wooden clogs, making the elements of the bridge supporting structure had been pulled across the Strait, ends of the cables were reeled out to winches on both shores and the cables were pre-tightened. Next the wooden clogs were separated so that the cables could slip down between them into the water. Then clogs, cut athwart according to the width of bridge were laid, thus making the platform. All constructional elements (cables, clogs placed lenghtwise and crosswise) were joined again. A six-foot high (1.77 m) palisade made of logs was placed on either side. Points of junction between transversely placed clogs were sealed and newly created irregularities were levelled with soil. Fig. 1 illustrates respective stages of bridge construction.

The way in which the bridge was constructed made water transport between the Aegean Sea and the Black Sea in the section between the bridges impossible. There was still a possibility of reaching the bridge track by sea on both sides.
Assuming that the oblong clog was one and a half feet (44.4 cm) in diameter and cables were one foot (29.6 cm) in diameter, the width of bridge deck on both bridges was 20 feet (5.92 m).

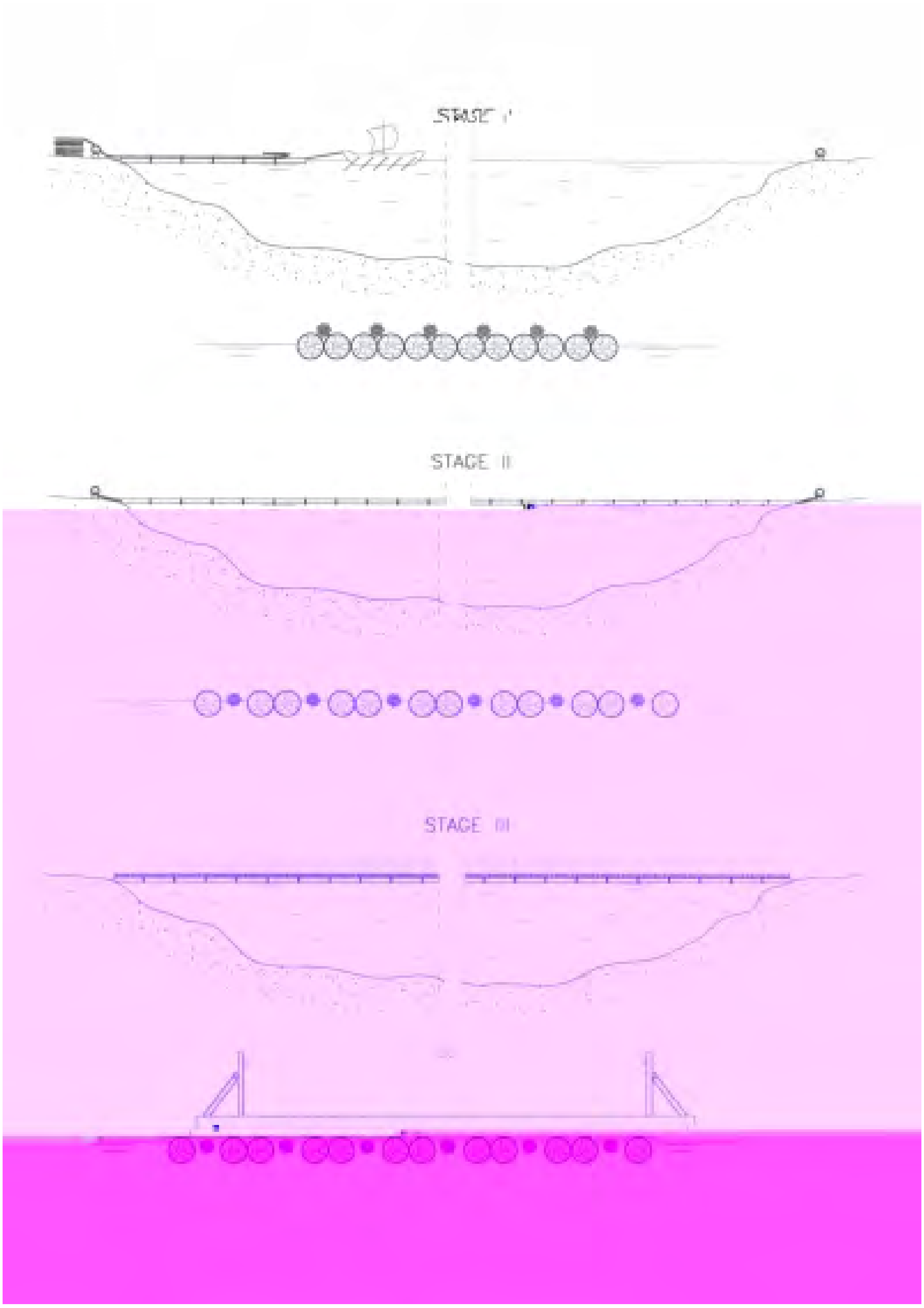

Fig. 1 Stages of bridge construction

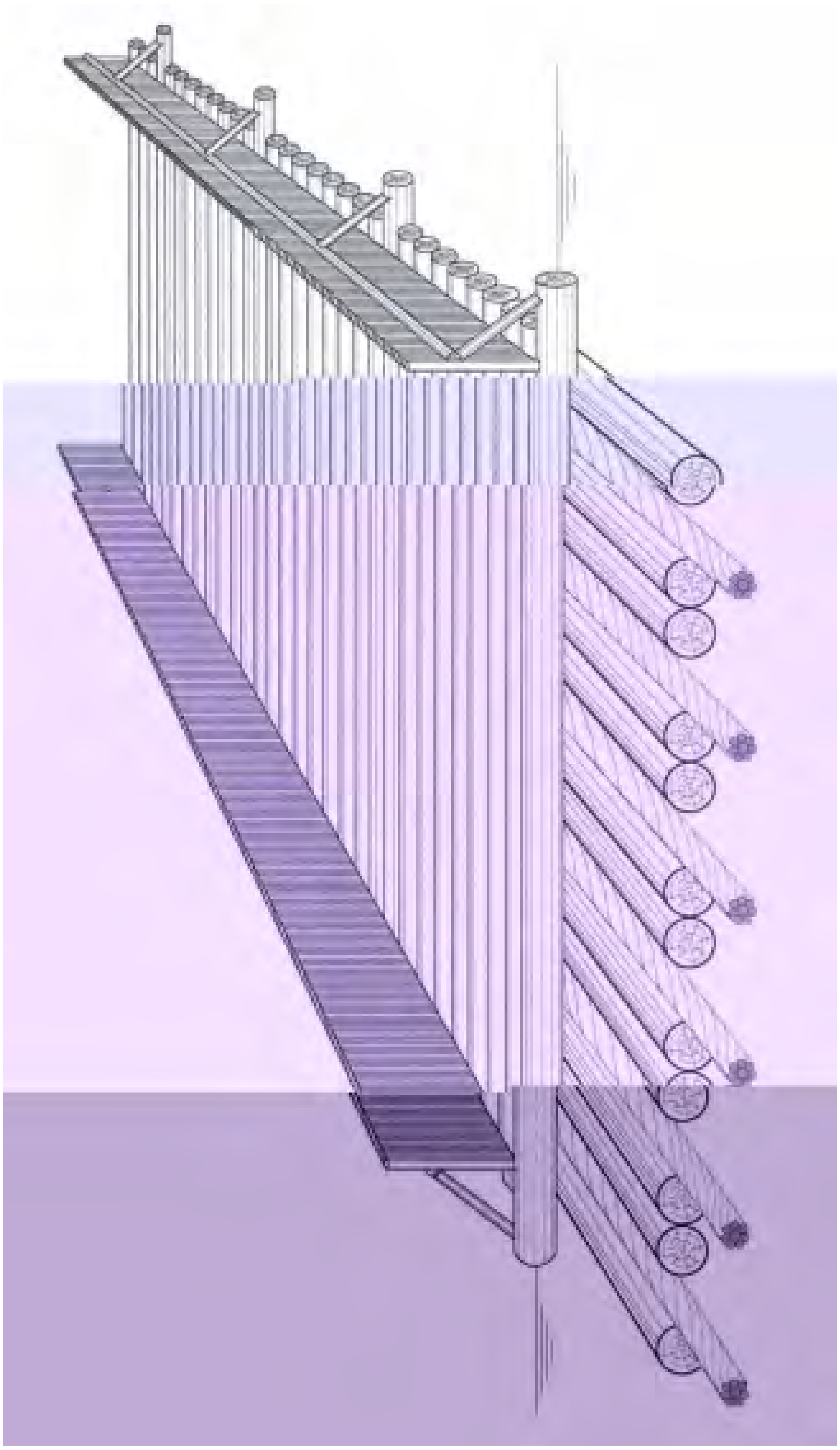

Fig.2 Construction of the bridge track across the Hellespont
according to the author

Numerous scholars are right to think that Herodotus considerably overestimated the size of the Persian Army. Hammond [2] says that the army could amount to 500 000. Even if we take for granted, following Herodotus, that the numerical force of infantry was 1 700 000 soldiers and that:

- the troops were walking in column in groups of six at intervals of two meters,
- at a speed of 3.5 km per hour,

it was possible for the army to cross within 7 days and nights.

8. RESULTS

According to the author's hypothesis, the bridges that Xerxes built across the Hellespont were floating bridges of the raft type [Fig. 2]. In this type of bridges cables made a stabilizing element which was to hold rafts in a position, and the ships were used as a protection against the destructive effect of sea waves. However, in all manuals and encyclopaedias the crossing is described as bridges whose supports are made by ships upon which a deck suspended on cables is situated.

In confirmation of the hypothesis presented, let us recall a fragment of Herodotus' work. And so, when Xerxes intended to come back from Europe to Asia, he ordered "the commanders to lead the ships out of Farlos heading speedily for the Hellespont in order to guard the bridges so as the king could go over them" [3, VIII, Ch. 107].

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IRGANDI BRIDGE FROM THE PAST TO THE FUTURE

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ABSTRACT

Irgandi Bridge, located on Gokdere river in Bursa, is one of the most extraordinary bridges in the world. Merchant Hoca Muslihuddin had it built in the form of a market in 1442 during the reign of Sultan Murad 2nd. Although Evliya Celebi who visited Bursa in 1640 wrote that there were 200 shops on the bridge, we know that in reality there were 32 shops on the bridge, 16 being located on each side, with one of them on the east being reserved as a small mosque. There were also inner spaces covered with vaults which were used as stables and storage on both sides of the single arch of the stone bridge. The bridge was partly demolished during a flood in the 18th century, and it was greatly affected by the severe earthquake that took place in Bursa in 1855, after which the shops on the bridge were constructed in the form of small wooden houses. Therefore, the stone bridge which is seen in drawings and engravings before 1855 changed greatly after this period as reflected in the 1862 Suphi Bey map and in photographs taken after 1855. The Irgandi Bridge was bombed and demolished while the Greek military forces were leaving Bursa in 1922 after the Turkish Independence War. The main vault of the bridge was reconstructed in concrete by the Governor of Bursa in 1949; however, the original shops on the bridge were not built. Research demonstrates that the length of the bridge was shortened and its height was raised in the process of certain urban planning decisions in Bursa. At present, there are attempts to restore the bridge in its original form; however, due to earthquake regulations it has been reported that it is impossible to restore the bridge in its original construction system and materials. The aim of this paper is to analyze the history of Irgandi Bridge and to evaluate the proposals for its restoration.

1. INTRODUCTION

Irgandi Bridge is located on Gokdere river in Bursa, which is one of the two main rivers directing the water from the slopes of Uludag mountain to the Bursa plain. However, it is different from the other bridges on Gokdere because it was built as a closed stone bridge comprising shops lined up in a row on both sides, and inner spaces used as stable and storage spaces. There are other examples of bridges in the world, like Ponte Vecchio in Florence Italy and Malabadi Bridge in Diyarbakir Turkey, comprising either retail areas above or spaces used as inns or storage inside. However, Irgandi Bridge is unique because it was planned as a complete structure with shops above and stable, inn and storage spaces inside [13]. The bridge, which was constructed in 1442 connecting the two sides of Gokdere valley in the northwest and southeast direction, experienced many transformations in its history, finally being repaired in concrete as a simple bridge without any shops in 1949. Since 1988, attempts have been made to restore this bridge and measured drawings, restitution and restoration projects have been prepared with this purpose. The aim of this paper is to analyze the history of Irgandi Bridge and to evaluate the proposals for its restoration.

2. HISTORY OF IRGANDI BRIDGE

In order to understand the construction process of Irgandi Bridge, the Ottoman social and economic structure should be analyzed. According to the established rules in the Ottoman State, it was very difficult for individuals except the Sultan and his close associates to gain economic power. Even the right to own property for the close associates of the Sultan was not free. In return for owning property, the associates of the Sultan had to provide soldiers named 'sipahi' from their regions to join the army in case of war. Furthermore, they were expected to establish pious foundations called 'vakıf' for the construction of public buildings according to the Islam rule of accomplishing charitable deeds. However, wealthy merchants, who played an important role in the trade relations of the Ottoman State, are also known to have contributed to the construction processes in the Ottoman period [15].

Bursa, which became the first capital of the Ottoman State in 1326, was not exempt from these processes. Merchants, who had a great impact on Bursa's becoming one of the most important trade centers in the world in the 15th century, also got involved in the construction processes, and had mosques, dervish lodges ('tekke'), recluse cells ('zaviye'), and commercial buildings established. Irgandi Bridge, which was constructed as a closed commercial center ('arasta') comprising shops lined up in a row on both sides, is one of these. It was built in 1442 by merchant Hoca Muslihuddin, the son of Irgandi Ali, who was one of the most important merchants in Bursa in mid 15th century and who gained considerable wealth as a result of the sale of silk from Azerbaijan to Italian merchants. The architect of the bridge is claimed to be Timurtas, the son of Abdullah [4].

Since the bridge experienced many transformations in its history due to various reasons in the past, the information about the bridge is available in written documents, especially books of travels, in engravings and in drawings accompanying the notes of past travelers (Figure 1) and in the photographs taken after mid nineteenth century. In the Seyahatname of Evliya Celebi, who visited Bursa in 1640, it is reported that there were 200 cotton and wool fluffer ('hallac') shops on the bridge. Evliya Celebi also stated that the name 'Irgandi' comes from the Turkish verb 'irgalanmak' which means 'to swinge, to shake'. This relationship was based by Evliya Celebi on a folk tale according to which an Ottoman warrior experienced the shaking of the ground when he hit the ground with a cleaver while he was on the way to the public bath and saw gold coins in the river, after which Orhan Ghazi advised him to spend this money on the accomplishment of a charitable deed.

Figure 1- An engraving of Irgandi Bridge by de Sinety in early 19th century

The warrior carried the treasure to his house, paid the necessary tax to the state, and had this bridge built with the rest of the money [9]. Richard Pockocke, a British traveler who visited Bursa in 1745, reported that Irgandi bridge, over which shops were built, was 90 steps long and 16 steps wide [9]. Miss Perdoe who stayed in Bursa in 1836 not only described the bridge, over which silk production shops were constructed, as a street over a river, but also made a sketch of the bridge which is an important source of information about the appearance of the bridge at the beginning of the nineteenth century. Charles Texier, a French traveler and archeologist who made research in Anatolia and Persia from 1833 to 1843 for the French Government, came to Bursa in 1839 and wrote that this bridge, which connected Moslem and Armenian neighborhoods, had a roof resembling some of the bridges in Switzerland. Texier also included an engraving about the bridge in his book [13]. Dalsar indicated that this bridge was the administrative center of the silk textile production guild, based on his research on

Canonical Records ('Şer'iye Sicilleri') [7], and this position was supported by Western travelers who visited Bursa in the 19th century.

There is conflicting information about the length of the bridge in different sources, varying from 45 to 300 meters. However, when the number of shops and their measures are taken into consideration, the length of the bridge can be calculated as 62.50 meters, and the width as 11.40 meters. The stone bridge, over which 32 shops, 16 on each side, were constructed, had a single arch with inner spaces on both sides of the arch, used as stable and storage area. According to the Ottoman Canonical Records which are 950 volumes, each with 500-700 pages including 25-30 lines, the bridge was donated to a pious foundation in 1558 (966 Muslim Calender) by Haci Muslihuddin, the grandson of merchant Hoca Muslihuddin who had the bridge constructed. Five of the units on the northeast side of the bridge, one of which was used as a small mosque, were reserved to bring revenue to the trustees of the pious foundation. The rents of the remaining 26 shops would be spent for prayers to the grandfather who had the bridge constructed and the rest would be sent to the poor in Medine. The shops on the bridge were let for 2 'akce's (the basic unit of the Ottoman money system) per day in early days. With the revenue gathered, the shops and the lead covering on the shops were renewed, and also some sidewalks in Pinarbasi and some bridges in Sakarya and Edirne were repaired [3].

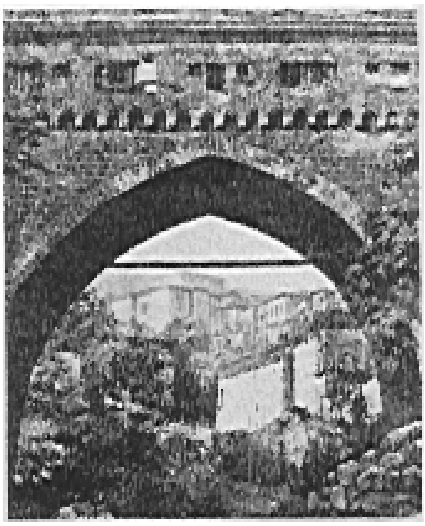

Figure 2- Photograph of Irgandi Bridge by Tremaux, P., 1854.

Irgandi market bridge, whose original walls were built from stone, had wooden gable roof construction on top of which lead covering was applied over gum lac. Later, after the lead covering collapsed in the 17th century, tiles were mounted in its place [4]. The drawings and engravings related with the bridge during the first half of the 19th century show that the market bridge had a high gable roof covered with tiles in the middle, and the shops with lower tile roofs on both sides. There are air and light vents on the roof, and the exterior walls of the shops rest on the single arch of the bridge with a moulding consisting of small arches. Shops have windows overlooking Gokdere on both sides, and small loopholes can be observed below the street level on both sides of the vault (Figure 2). Various sources mention that the market bridge was closed in the evenings with large iron doors at both ends. There were also two doors with pointed arches for entering the inner spaces below the street level, on the northeast side of the bridge originally; however, only one of these doors is existent at present. It is probable that the door on the southeast side has been removed in the process of shortening the bridge during a repair at an unknown date [14] (Figure 3). These spaces, ventilated by loopholes, and containing beautiful tile ceiling decoration, were probably used first as a stable for the horses and camels of the merchants who bought and sold goods in the market above, and as an inn for the caretakers of these animals, and later as storage and production area. The inner space on the northwest had a rectangular plan with dimensions of 4.50 x 14.85 meters, with a low vault ceiling [13]. Further research by the staff of the Metropolitan Municipality of Bursa in 1990 has revealed that a similar space covered with a vault existed in the east side of the bridge. As a result of this research, an estimation about the east and west end walls of the bridge has also become possible [4].

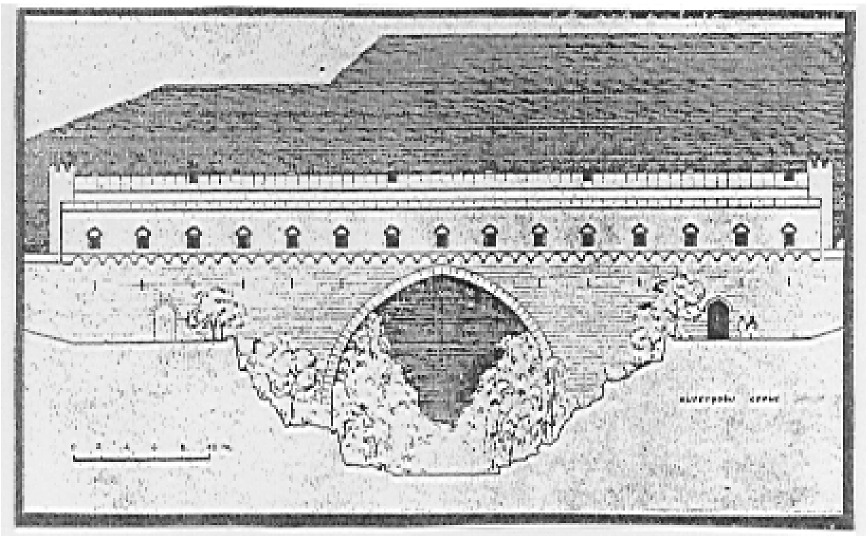

Figure 3- Restitution of the Northeast Elevation by Onge.

There were also 24 small cells used as storage spaces inside the bridge below the street level, each lighted with a small loophole. These cells, which were connected to the shops above with vents, were divided with walls in accordance with the dimensions of the shops above. Research shows that not all of these cells were open to the stable spaces, but rather that necessary connections were made with the purpose of ventilation. The walls of the cells and the stable areas were constructed in stone until the bridle iron level and in brick above this level (Figures 4, 5).

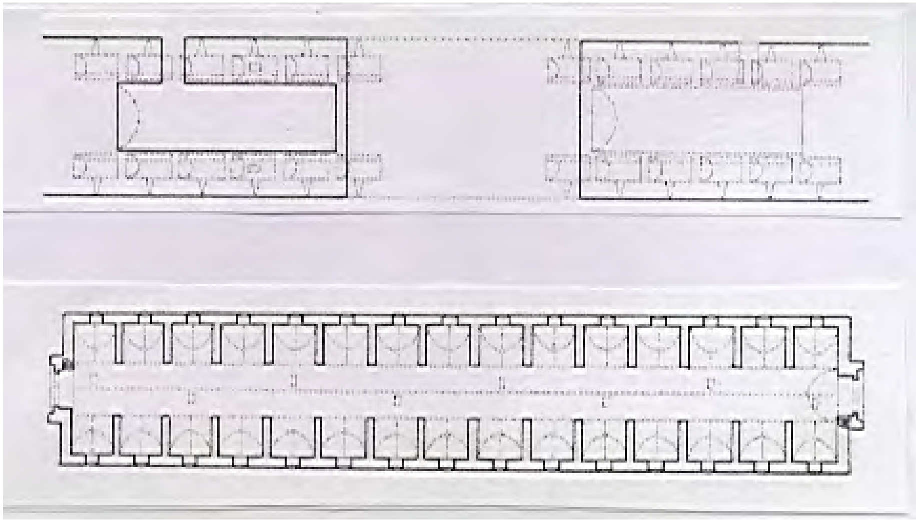

Figure 4-Restitution of the lower and upper levels of Irgandi Bridge by Onge

Figure 5- Inner Perspective of Irgandi Bridge According to the Restitution Project

Irgandi Bridge was partially demolished during a flood in the 18th century, and it was severely affected by the 1855 earthquake in Bursa. This earthquake which took place on January 31, 1855 around 15:15, many monumental buildings, including Ulu, Green, Yildirim, Orhan, Sehadet, Hudavendigar and Muradiye Mosques, and residential areas in Bursa were severely damaged [11]. The effect of the earthquake on Irgandi bridge is described by Cevdet Pasha in Tezakir [5]. Cevdet Pasha explains that Irgandi Bridge was totally demolished as a result of the striking of large rocks from Uludag which were dragged by Gokdere river. Although this description is usually considered as exaggerated [13], it identifies the reasons for the transformation of the bridge after 1855. In fact, the photographs taken in the second half of the 19th century demonstrate that the closed stone market bridge was replaced by wooden shops resembling houses, lined up on both sides of the bridge, which has now become an open market. A photograph taken by Berggren in 1880 shows that there are cavities among the shops, whereas photographs taken in late 1890's reflect the bridge as completely filled with shops (Figures 6, 7). The 1862 Suphi Bey map of Bursa demonstrates this phase of the bridge (Figure 8). The stone structure on the bridge, which was severely damaged by the earthquake in 1855, must have been removed in order to prevent the bridge from further loads and a lighter construction from wood must have replaced the old structure.

Figure 6- A photograph of Irgandi Bridge by Berggren in 1880

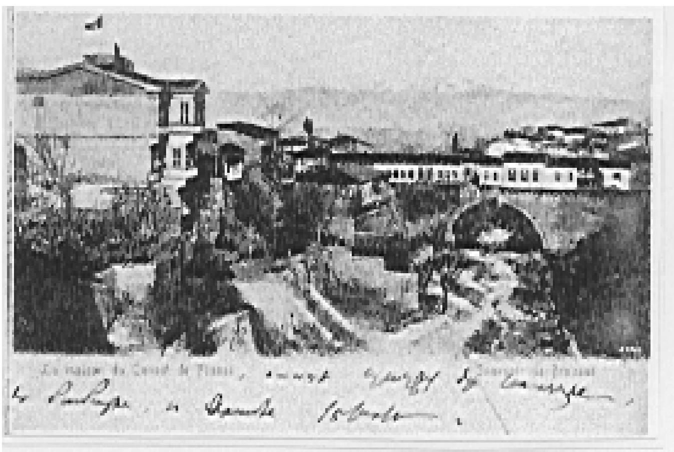

Figure 7- A photograph of Irgandi Bridge taken in late 1890's

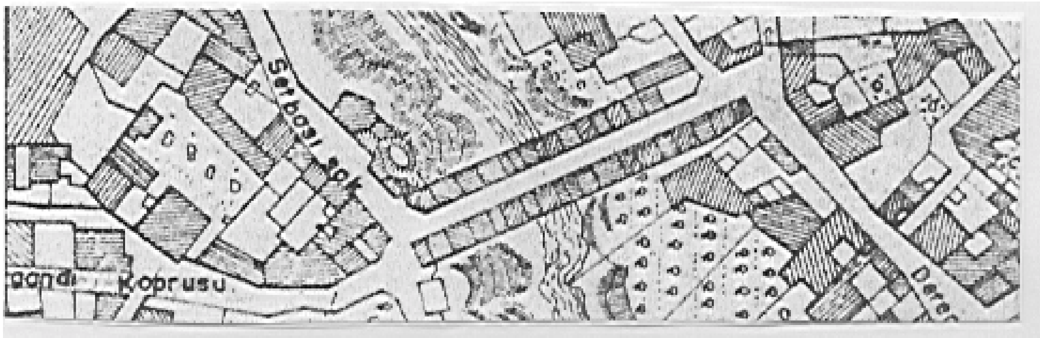

Figure 8- Plan of Irgandi Bridge in 1862 Suphi Bey Map of Bursa

The bridge, which was bombed by the Greek military forces as they were leaving Bursa in 1922 after the Turkish Independence War, was repaired by the Bursa governor Hasim Iscan without any shops in 1949 as a result of the request of Kazim Baykal, the chairman of the Bursa Historical Artifacts Society. In this phase, the main vault of the bridge was reconstructed in reinforced concrete, and stone arches were built on both of its sides (Figures 9 and 10).

Figure 9- A photograph taken after the bridge was bombed

Figure 10- A photograph taken by Gabriel after the bridge was repaired in 1949

A comparison of the present situation of the bridge with its old photographs reveals that the reconstructed bridge is 60 cm. higher than its original level as a result of various street covering processes in different time periods, and that the southeast side is 120 cm. higher than the northwest side. Furthermore, the bridge has been shortened 920 cm. on the southeast and the arched storage door on this side has been removed. These changes are related with rapid urbanization processes in Bursa, as a result of which Gokdere valley has been narrowed down, and its slopes have become steeper as roads and buildings spread towards the valley [14]. In fact, Gokdere Boulevard passes from the southeast side of the bridge where Kurdoglu Cemetery existed originally. When the 32 shops are placed on the bridge according to the frequency of the loopholes and the dimensions of the cells in the bridge, it is found out that the bridge originally extended until the center of the road which passes from the southeast side of the bridge at present. There are also problems on the northwest since some houses have been built adjacent to the bridge in later phases.

2. PROPOSALS FOR THE RESTORATION OF IRGANDI BRIDGE

The first record of an attempt to restore Irgandi Bridge in the Bursa Cultural and Natural Wealth Preservation Council is dated 26.11.1988. The members of the Council approved the requests of several institutions for the restoration of Irgandi Bridge in their meeting on 26.11.1988. In their 42nd meeting on 12.11.1989 , the members of the Council decided to have those parts of the bridge which have been covered with earth cleared by the staff of the Bursa Museum in order for healthy measured drawings to be prepared. This decision was accompanied by a protocol signed on 11.11.1989 among Bursa Cultural and Natural Wealth Preservation Council, Bursa Pious Foundations Regional Administration, Metropolitan Municipality of Bursa, Yildirim County Municipality, Osmangazi County Municipality, Bursa Historical Artifacts Society Presidency, Bursa Museum Administration, Chamber of Architects Bursa Section and Prof. Dr. Yilmaz Onge. According to this protocol, each of the related parties were assigned certain duties for the restoration of Irgandi Bridge.

On 14.09.1991, Bursa Cultural and Natural Wealth Preservation Council evaluated the research carried out by the related parties that signed the protocol in 1989 and arrived at the following conclusions: a) the covering and infill materials could be removed in locations suggested by Prof. Dr. Yilmaz Onge on the bridge and at the endpoints of the bridge in order to determine the cell axes in the understructure of the bridge and to prepare healthy measured drawings of the understructure; b) exploratory wells should be dug in order to identify the cast concrete and reinforced concrete addition to the vault of the bridge when it was reconstructed in 1949 and projects and proposals prepared according to the results of these exploratory wells should be brought to the Council; c) repair and strengthening of the understructure of the bridge should be started after the completion of the above activities; d) studies related with the restoration project should be continued while the exploration, project and repair of the understructure were being carried out; e) proposals for the restoration of the bridge should be prepared with a contemporary approach based on the functions to be assigned to the bridge and on the available information on the bridge. This decision of the Council was criticized by Kazim Baykal, and some others who participated in this meeting because they believed that rather than the application of modern techniques, the historical development of the bridge should be respected and that the veneer stones, materials and later workmanship should be considered as reflections of the historical phases of the bridge [4].

After this meeting, Osmangazi Municipality Council decided to have the measured drawings, restitution and restoration projects of Irgandi Bridge prepared on 25.10.1994, and transferred this job to M. Kemal Altan-Semih Tuncer Architecture, Engineering, Counciling Firm on 11.11.1994. In the meanwhile, Osmangazi Municipality applied to the Bursa Pious Foundations Regional Administration on 14.11.1994 for acquiring the utilization right of the bridge in return for restoring the bridge. The final protocol between Osmangazi Municipality and Bursa Pious Foundations Regional Administration was approved by Pious Foundations General Directorate on 11.08.1998 on the basis of restore-operate-transfer model, and the utilization right of the bridge was handed to Osmangazi Municipality for 10 years.

In the restitution project prepared by M. Kemal Altan-Semih Tuncer Architecture, Engineering, Counciling Firm, the original dimensions of the bridge are 62.50 x 11.40 meters and comprises 32 shops. However, due to various transformations in the vicinity of the bridge in history, the length of the bridge has been shortened as reflected in the measured drawings. As a result, the number of shops in the restoration project had to be reduced to 24 at first, and later to 22 after the completion of Gokdere Boulevard. The projects were sent to Bursa Cultural and Natural Wealth Preservation Council on 19.06.1995, and the Council accepted these projects in its meeting held on 09.01.1998. However, after the details of the restoration project were submitted to the Council, certain problems arose in relation to the utilization of skeleton construction system and the choice of reinforced concrete and gas concrete in the reconstruction of the bridge.

According to the Principle Decision no. 660 declared by the Turkish Ministry of Culture Cultural and Natural Wealth Preservation High Council in 05.11.1999, the reconstruction of a registered structure has to be realized in its original site and construction area, respecting its original elevation characteristics, mass, height, plan scheme, materials and construction techniques based on a detailed restitution study [10]. In its evaluation of the above mentioned principle, Bursa Cultural and Natural Wealth Preservation Council asked for additional reports from the related Department of a University confirming the choice of the structural system and materials proposed in the restoration project of Irgandi Bridge. A report from Middle East Technical University Department of Civil Engineering Structures Division, which was provided after this decision, explained that since the center of the bridge was damaged previously, the utilization of original stone material which is weak against tension would not be correct. According to this report, the loads have to be rather transferred with a beam system to the ends of the bridge. The report also emphasized the fact that the additional upper structure would bring extra loads to the bridge which is located in a first degree earthquake zone. The use of original stone material would mean that a bearing construction system would have to be applied resulting in a heavier structural mass which would be unsuitable for the durability of the bridge. The conclusion of the report was that a reinforced concrete skeleton structural system has to be chosen for the reconstruction of Irgandi Bridge.

3. CONCLUSION

Bursa Cultural and Natural Wealth Preservation Council has still not given its final decision on the restoration of Irgandi Bridge. If the Principle Decision no. 660 is applied literally, this bridge cannot be reconstructed because the new bridge has to be shorter than its original length due to the construction of roads along Gokdere river and various houses one of which on the west end of the bridge is registered as a an example of civil architecture. The detailed study of the bridge by the Firm who prepared the measured drawings, restitution and restoration projects, and the report from Department of Civil Engineering at M.E.T.U. reveals that original construction system and materials cannot be used in the reconstruction of the bridge due to earthquake regulations.

The question is to whether to leave the bridge in its present state or to reconstruct the bridge in a way which best reminds its original form. I believe that the second alternative has to be chosen as a reflection of the respect for cultural heritage. Interpretations of the Principle Decisions should be possible in such extraordinary cases. One of the solutions in this situation would be the reconstruction of the ancient structure with a completely new approach, using contemporary techniques and materials in order to demonstrate deliberately that the reconstruction belongs to the present. This would mean an interpretation of the original elevation characteristics, mass, height and plan scheme of the bridge. Another solution in this case would be to reconstruct Irgandi bridge as a copy of its original form with new construction system and materials, as confirmed in the structural report of the bridge, hidden behind veneers resembling the original materials. In this case, the original length of the bridge should be reminded either in two dimensions as a street surface pattern or in three dimensions as elements extending over the newly constructed roads. Whatever solution is chosen, the aim should be to enliven memories of the past for future generations. As Sir Geoffrey Jellicoe remarks in the early 1900's, "Architecture is to make us know and remember who we are".

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THE RILA MONASTERY IN THE LIGHT OF BULGARIAN HISTORICAL AND CULTURAL BUILDING HERITAGE

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ABSTRACT

The proposed article presents the attempt of the authors to describe the biggest monastery situated on Bulgarian lands dating back to the Renaissance period. The Monastery was found by St. Ivan Rilski and represents a closed complex of massive outer stonewalls, erected in the form of an irregular quadrangle. Within its internal space are located the Hrelio's Defensive Tower dating back to 1334 and the Church "Holy Virgin" dating back to 1834. Subject of this paper is their architectural and structural execution, as well as the quality of their preservation during the past ten centuries.

1. INTRODUCTION

The people, who inhabited the lands of Bulgaria, have left an abundant cultural heritage with respect to architecture and buildings. For more than 1300 years the people living in this country, which had been founded in 681, were creating their own culture. Nowadays material cultural monuments show the course of a remarkable human and national history. As part of Bulgarian national architectural heritage monuments of different historical ages have been preserved. Some of them date back to 25 centuries ago. They are an expression of the cultural achievements and creativity of Bulgarian people in the course of centuries and show abundant traditions in the field of architecture and building.

The Rila Monastery has a special place among cultural monuments. The Monastery consists of a Central Monastery Building, the Church "Holy Virgin" and the Hrelio's Defensive Tower. The monastery complex, which is considered a relic in Bulgarian culture and history, was recognized as a monument - part of the worlds cultural heritage in 1985 and since then it is under the auspices of UNESCO [1].

2. DESCRIPTION OF THE RILA MONASTERY

The Rila Monastery is the largest monastery complex on Bulgarian lands dating back to the Renaissance Age in Bulgaria. It is considered the most remarkable achievement of Bulgarian builders in this period both with respect to its architecture and its structure. According to historical sources the Monastery was found in 927-941 by St. Ivan Rilski. The Monastery is situated on the southern slope of the Rila Mountain at 120 km from Sofia. In the course of centuries the Monastery passed through several stages of reconstruction. It was burned down several times and rebuilt again. The present Central Building was re-constructed in 1816-1847. In 1960-1964 the east wing was re-built with a new structure. The Monastery was erected as a closed complex of buildings, surrounding an inner yard in the form of an irregular quadrangle (Fig. 1).

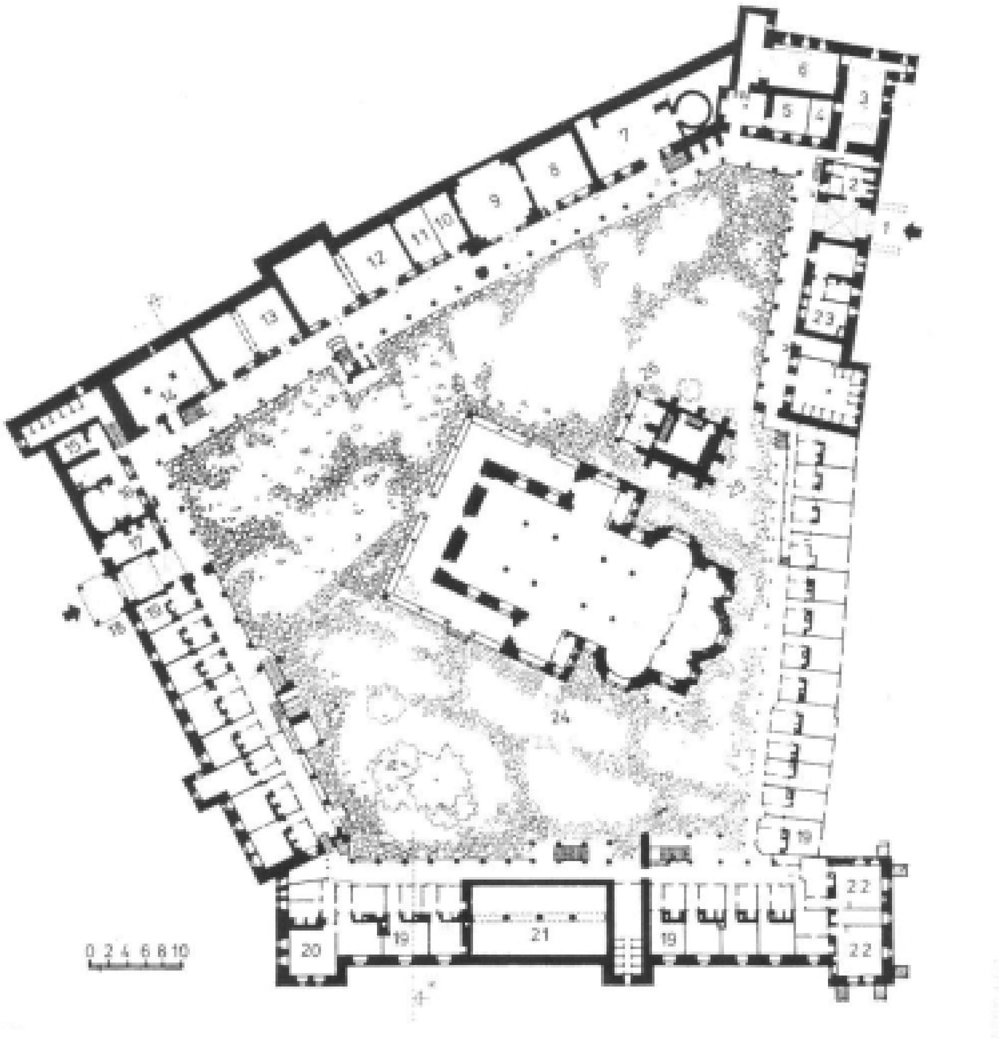

Fig. 1

Its total area is 8800 m^2. The outer architecture has the characteristics of a fortress. The walls were built by using stone masonry and have window openings. The width of the walls varies in proportion to their height from 1.6m at the foundations to 0.8m at the top (Fig.2).

The front elevation consists of two main elements- arches and columns (Fig. 3). The inside walls have timber structures filled with brick masonry. The floor and roof structures are made of timber. The roof was repaired in the twenties. The Central Building has six storeys, two of which are underground. The monastery housing wings have four storeys and contain more than 300 monastic accommodations, 4 chapels and numerous guest-rooms and store-rooms. The most interesting of all premises is the large monastery kitchen, called “magernitsa”. It is a massive tower in the form of a pyramid, which passes through all floors and ends up over the roof in a dome. It was erected on 4 massive arches on a square base and reaches the height of 22m. The pyramid structure was built due to 10 consecutive rows of arches arranged over one another on a base in the form of an octagon (Fig.4).

Fig. 2

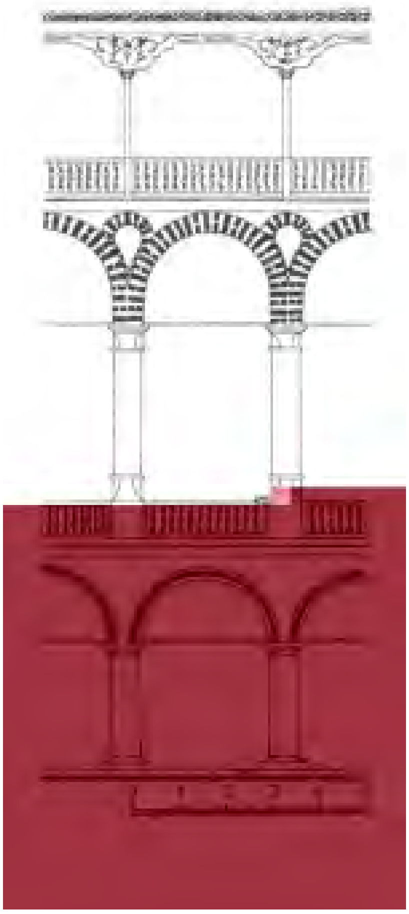

Fig. 4

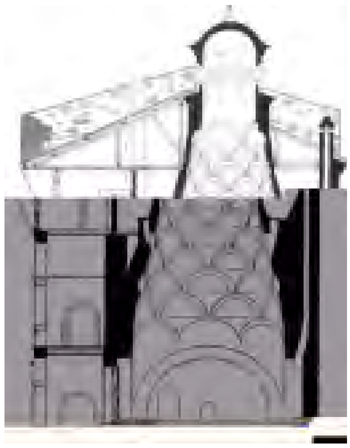

Fig. 3

Fig. 5

With its structure, tectonics, spatial solution and architecture this part of the Rila Monastery is a unique achievement (Fig. 5).

3. DESCRIPTION OF THE CHURCH “HOLY VIRGIN”

The Church “Holy Virgin” in the Rila Monastery was built in 1834-1838 in the middle of the monastery yard. With its layout, design and front elevation solutions, the church represents an astonishing achievement of Bulgarian architecture during the age of the Renaissance [2]. It is a five-domed building with two lateral chapels and a gallery opened to the West, to the North and to the South (Fig. 6).

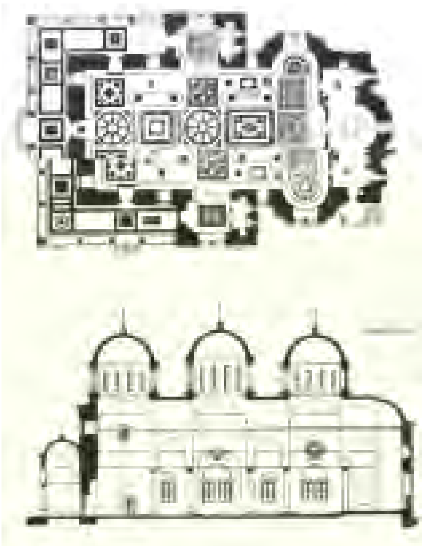

Fig. 6

Three large domes with high drums form the axis of the main space of the church, which has the impressive dimensions of 14/31m. Wide-span arches at the transverse axes of the domes create a complex cross-like composition. The one-storeyed arched gallery is interesting with its unequal spans between the columns and blind domes at the roof. The western wall of the church bends into a triple yoke-shaped gable, which forms the main cornice of the building (Fig.7). The walls of the church were erected by using layers of stones and bricks. The complicated architectural and structural composition of the Church “Holy Virgin” represents the emphasis in the whole monastery complex. This remarkable religious monument is an integral part of the thorough harmony of the monastery complex (Fig. 8).

Fig. 7

Fig. 8

4. DESCRIPTION OF THE HRELIO'S DEFENSIVE TOWER

The oldest building preserved in the historical Rila Monastery is the Hrelio's Defensive Tower, which belonged to the feudal landlord Hrelio. The Rila Monastery was situated in his estate. The defensive tower was erected in 1335. Its purpose was to protect both feudal lords and monks from the attacks of enemies [3]. The outer architecture of the Tower is austere and imposing (Fig.9).

Fig. 9

The walls are 1.8m wide and filled with crumbled stone and lime. They are supported by massive stone counterforces uninterrupted along their length. The counterforces are connected at the top by arch structures (Fig.10). In the walls under the arches the builders made wide openings for defense purposes. The Tower resembles a fortress. There are crenelles and a platform at the top of the Tower. Its total height is 23m. The Tower has 6 levels. A staircase of stone built in the wall served for ascension. As far as layout is concerned the Tower has square dimensions. There is a ground floor with an arch-formed ceiling. A chapel with a domed ceiling with 6 ribs was built in the top floor. The floor structures between storeyes were made of timber. The emphasized vertical articulation adds dynamics and plasticity to the thorough appearance of the monument. Finishing the counterforces by arches presents an interesting architectural approach unique for the period of construction. Bricks were used only for decoration of arches over the counterforces and the corners between them (Fig.11).

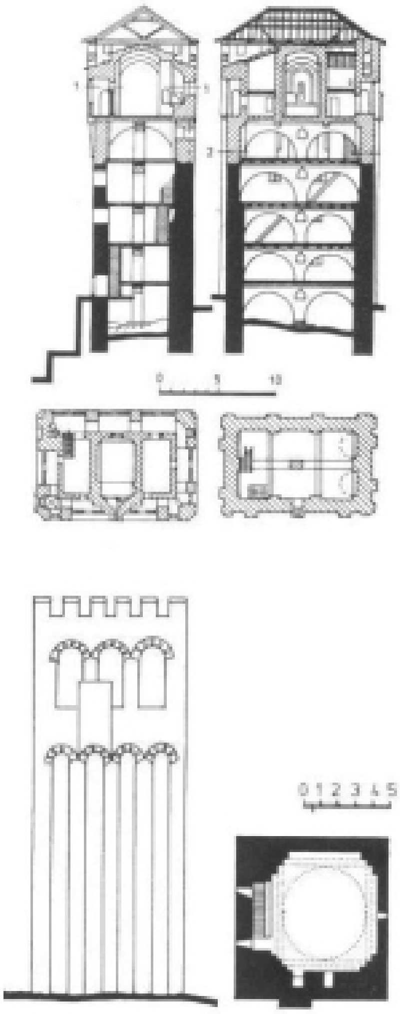

Fig. 10

Fig. 11

5. DESCRIPTION OF THE SEISMIC MONITORING SYSTEM

Bulgarian government takes various measures aimed at preserving Bulgarian cultural heritage, which holds Bulgarian national spirit and national identity. One strategic line is to finance research work on the reactions of respective structures to seismic forces.

The following paragraph is aimed to present the approach and the results of a research performed at CLSMSE-BAN. The objective of the research was to determine the dynamic characteristics/parameters of the Rila Monastery building, as well as the church "Holy Virgin" and the Hrelio's Defensive Tower [4].

The dynamic characteristics were determined in result of an analysis of registered real earthquakes, regenerated by a system for seismic monitoring. The latter had been created by using equipment donated by UNESCO. The equipment involved consists of 4 digital accelerographs, produced by the Swiss company GEOSYS. The digital accelerographs include an accelerometer block SSA-20 and an operational computer block GSR-20. Three independent seismic channels in SSA-20 register three perpendicular movement components. The operational computer block is a 12-byte system for recording seismic data.

The seismic monitoring system ensures:

- Registration of seismic signals, featured as input signals for the respective structure
- Registration of movements of specific structural points in result of the impact of seismic forces.

Two earthquakes have been registered with the system described in the preceding paragraph. The earthquake on 03/07/1998 had a magnitude of M=3.7. The epicenter was at a distance of 27km to the Northwest from the Monastery with seismic focus at a depth of 13km. The earthquake on 10/09/1998 had a magnitude of M=3.1 and the epicenter was at 48km to the Northeast from the Monastery (Fig.12). Due to the analysis of the data recorded during the earthquakes the fundamental vibration period, the frequency (f_I=3.613Hz; T_I=0.277s) and the frequency spectra of amplification of the three structural components at level 1157m compared to the ground level (1139m) were determined. The dynamic characteristics obtained could be used for analysis of seismic loading.

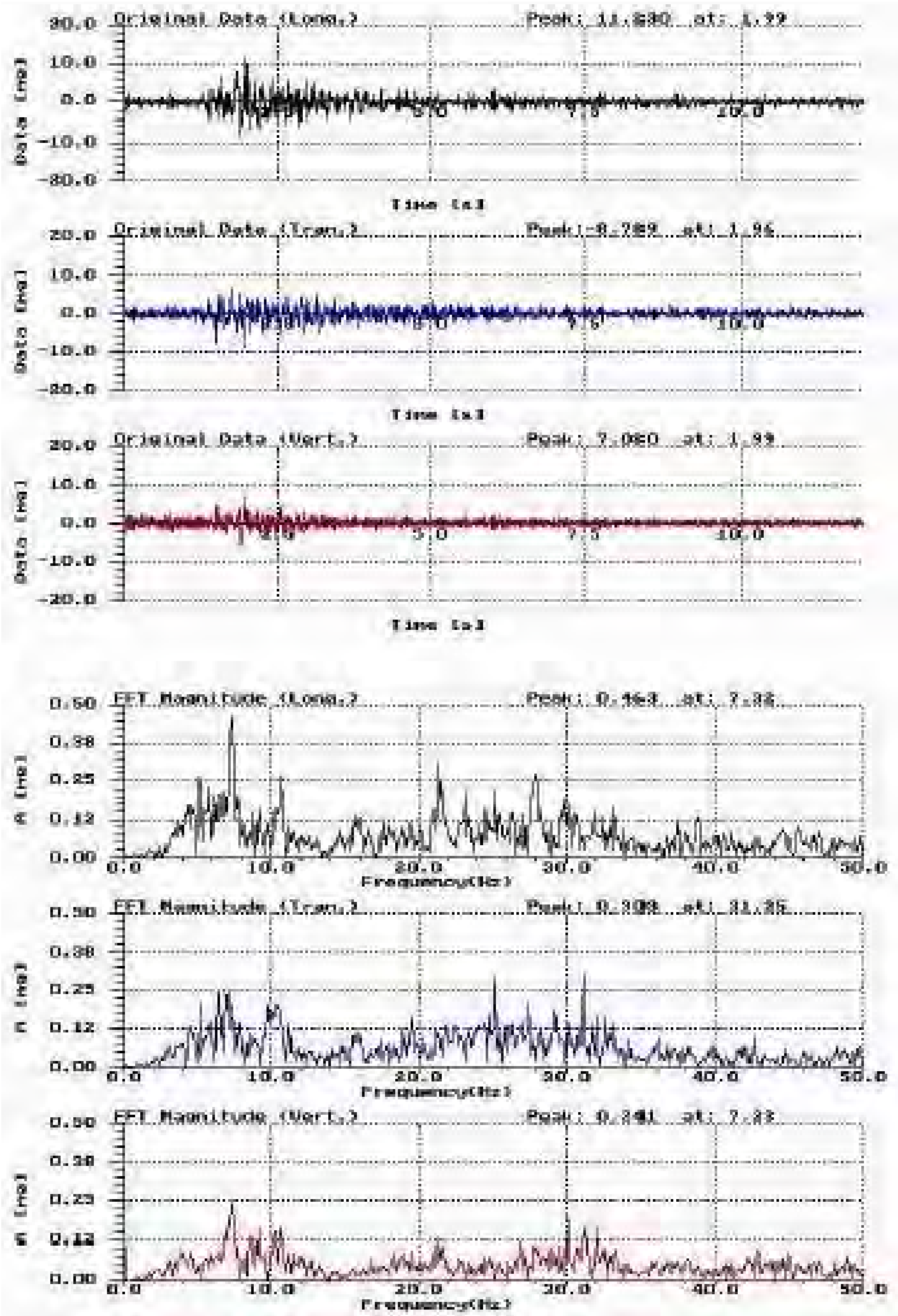

Fig. 12

6. CONCLUSION

The history of the Rila Monastery is a unique expression of the life, the philosophy, the views and the insights of Bulgarian people living in this specific period in history. The Monastery was built on a small plain in the folds of the Rila Mountain with a boundless panorama of imposing massifs, rocky mountain peaks and venerable forests. Nowadays Bulgarians preserve the Rila Monastery and honour it as one of the most precious monuments of Bulgarian national culture and architecture from the past and from the period of the Renaissance in Bulgaria in particular.

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THE SACRED AND SYMBOLIC STRUCTURE OF FOLKLORIC ARCHITECTURE

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ABSTRACT

The folkloric architectural heritage represents a rather vast matter that touches the problems which nowadays become more and more actual and their goal is a layer identification with the materiel and spiritual heritage and realisation of the contacts with all the marks they carry. Thus, the question for determination of the character of the architectural space is defined by a multitude of demands which have to correspond to the various character of the problems and which are related to this matter.

1. ARCHETYPAL ENTRIES

At present times we are witnesses of the permanent process of impoverisation of the significance of the architectural space first of all because of the fact that its function is since long time only utilitarian one and deprived from every possibility to represent a more complex mytho-poetic structure which derives from the accumulated psychological, religious and social moments. The total desacralisation of the contemporary architectonic space has alienated from us all those layers of the traditional architectonic constructions that contained in them as a projection of specific symbolic and mythical valences not only of some particular constructions abut of whole urban structures too. Some contemporary tendencies accept the regression of the conception of the architectonic space as their own right, imposing it even as a form of culture.

In contrast to them, in the rural settlements has survived the feeling for the sacred since thanks to it “a certain Christianity has been retained, experienced as a cosmic liturgy”. Those settlements are still keeping the remembrances of the sacred places and trees in the memory of their inhabitants, they conduct certain acts for protection from the “evil forces” and they still subordinate their place of living to a whole system of rites, beliefs and rituals observed with the aim of respecting the sacrum and its reconciliation with the life through a definite practical activity.

Even there are such characteristic examples that highlight very clearly the thesis of parallel existence of the sacred and the profane and their permanent active conditioning. The two adopted models: the one that defines the sacred and irrational established as an order in the chaos of the religious projections and the other as an image of its own microcosms - the house in which take place all the segments of life. They actually are the “sacred” and the “profane” space which in their projection offer an identical image.

The phenomenon of psychologically unconscious participation in a definite cult with a common ideal becomes impossible today, but it is rarely charged with an ideological propulsive energy which would come out from the need for identification. The rituals are loosing their meaning and their sacral character and become everyday customs to which we are still clinging although we are not conscious about their motives for they are deeply repressed.

The archetypal entries have been confirmed in the expressive language of the traditional objects, confirming their identification in all layers. In those case, the exactness of some rational problems coexists parallel with the active and constructive thought from the other side of completely conceptualised kind of “consciousness” and knowledge that possesses properties related before all more to “mythos” than to “logos”, more to the iconic than to the semantic.

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3. THE PRIMAL IMAGINARY

The primal architectural structures represents very important part of the traditional architectural heritage. In the process of creating of the local specifics, this type of objects represents the archetype of particular elements. There is a presence of rational, as well as irrational - mythopoetic components in their structure.

This type of objects, exist not only on Balkan's, but in other countries and regions, pointing to the fact, that some traditional architectonic structures, survived in their original form . This type of objects was Isolated and untouched from the other influences during the process of creating and developing of the local styles and specifics. Quite opposite is the total "desacralisation" of the modern architectural space, as a process that destroys all the connections with the traditional background meaning that some architectural objects or urban structures are based only on a specific projections of symbolical and myth values. Regression of the comprehension about architectural space is taken by some modern theories as their own right, even intruding it as a specific kind of culture. As an opposite of this opinion, is the traditional context in which we are trying to determine the basic definitions that can make the connections with the tradition.

The traditional context is structured from different categories of meanings. They are taking place in the collective memory even like transformed models which were changed in their basic traditional code. Traditional context is structured by several components :

- *Myth consciousness (religious consciousness),* can still be recognised in the thinking and behaviour of the people who live in rural regions. It is manifested on different ways, especially in the cycles of customs, parts of some rituals, and practising some manner of actions nowadays treated as traditional. They are built in their everyday cycles as a structure which is supplementing the wider context of the comprehension of life and its basic components. The presence of that consciousness determine the connection of some individuals with all categories of the term *"place" (locus*).

Defined in that way, this "*place"* possesses all components which means survival and protection of life. This components looks very rational on first side, but, beside that, they represent a typical example of permanent corresponding between rational and irrational side, creating very complex picture of the life in that kind of rural structure.

Further researching had to answer the question how to establish the "new irrationality of space", or "new symbolism of space" . This has to be created on some specifics which can justify and explain different kinds of human communities and settlements. Living in them will establish new parameters for creating the social life of the individuals in it.

- *Symbols, their meaning, cosmisaton of the primal environment*

Traditionally, this kind of space is very seriously treated (very close to the religious opinion which says that the space first has to be"*marked*" and explained as a space with different type of values.) including the settlement and the particular objects too. In the process of introduction in the irrational sphere, some of the elements are developed to the level of symbol. The meaning of the sun, water, winds, orientation and others, in rational meaning defines the climate , but in irrational and mythopoetic way, they are defining different type of categories which are the base for building the customs and rituals as representatives of local groups and communities. This points to the fact that "the rural man" and his point of view, depend on establishing some kind of elementary "cosmization" of the space for living, as a basic structure for the settlement in the future. In this way, the basic natural factors in this process of cosmisation, are equal with the categories which belong to the irrational sphere. This is not an accidental note, because the picture of the city in the future, has to offer a model in which the process of solving the basic problems will become a "new religion". Only on that level, individuals will believe in it, and even try to identify themselves fully, because that is the way to make the theory true.

Cosmisation of the "place" for living as same as the process of establishing the "symbolical scale of values" in it, discovers and approves the cyclicity of the nature as a necessity which has to be involved in any structure. That need is immanent for the planing of the future, because in that way, the substance of the natural processes will be included during the planing and structuring of the new models. This models will be placed in direct dependence from the nature and its present condition.

- *The community as an archetypal human need*

The community, considered in its wider context, is defining the term "middle interest", meaning the symbolical place that is the centre of "gathering".

In traditional meaning, the community presents a form of selfdepending organism, based on the system that provides all levels of communication, and all levels of protection and survival. All the segments that we have mentioned, are affirmed through permanent exchange of experience and superstructure of the individual "modus of consciousness". They are structuring the collective consciousness and the archetypal need for belonging to specific group or community.

This specific kind of ”gathering" defines the functional organisation not only for the individual living structures. It offers the definition to the organisation of the multiple microstructures organised in a bigger organism that has its biorhythm in correlation with nature.

Different kinds of communities were always the main creator of interactive relationships. The process of transforming the connections between particular individualities inside the community shows the differences in the development of the understanding the term "middle interest". Its meaning is much more symbolical than rational and the archetypal structure of it, can show the main points of the future possibilities of defining the "centres", understanded as a connection point between the past and the future.

4. THE HOUSE

Symbolical values of the house and its components, are structured from the categories with different mechanisms and they could be discovered not only in the traditional forms of housing but sometimes in the modern architectonic structures too. In the process of understanding the architectural space as a category and phenomenon with many different meanings, we are getting into the logic of it's "internal" construction, and to the point of the "mythopoetic" specifics of the house. This permanent need of reactualisation of the meaning of these archetypes in modern conditions of living, have its main purpose : decontextualisation of the architectural heritage and revision of the presence theories. The experience we can get during the researching process, is very useful in developing of the architectural theory and praxis, so necessary this days, when the need of complex studies becomes a basic skill.

In the act of sacralisation of the place which is "transformed" into house we can find some specific components responsible for the "mythopoetical" feature of the house. Religious behaviour defining the concept of the house is structured with passive exeptance of totalities - absolute categories, according to the chrystianisation of comprehension. This features are not some kind of law, but "specific attributes, not their substance". Construction of the house, its "creation", presents a process of repeating the universal technology, which means that "metaphysical technology of the house is equal with the metaphysical technology of man". Cosmological level which is established with consolidation of the house as a metaphysical object, allows us to recognise its parts as metaphysical levels of single realities existing in a complex system of meanings defined before. Defined as "archetypal", this components are proving their identification in all levels and procedures of "creating" the house. Opposite the visible reality, "things" appears as "illusion" which has its own structure because its own legality. A different kind of consciousness comes through getting back to this legality..:"the picture has no recurrent effect on the spirit as independent creation - for the spirit, it becomes reflection of its own creation power.."

4. 1. The sacrifice

In the paradigmatic process of mediation between the "sacrum" and the reality, the sacrifice appear in many forms and substitutes. Variant forms of the motif of sacrifice as a feature builded in its structure (manifested through possibility man to become god and god to become man), could be found not only in the primal religions, customs and rituals, but in the basic forms of cultural religions. We can find the sense of this act and its real religious and speculative dimensions in cases where god is taken as victim or he gets himself as a victim. Here, we must determine the difference between the primal sincreticity of the elements of sacrifice and latest presentations of cosmogonical and athropogonical victim as a result of different and transformed process of thinking.

The successors of anonymous builders from our folcloric architectural tradition, today are still practising specific actions every time on the beginning with the building process. They are still bearing in mind the ancient fear that the house (the"world") will collapse without the blood of the victim (sacred lamb) on the first eastern stone (the first stone on the base). This "mnestic" traces are leading this actions through the sphere of mythopoetical transformation of the "real space" and through the symbolical transformation of consciousness.

The building victim as a specific kind of manifestation of some acts exists in many different cultures. This victims are based on transforming the chaos to cosmos-on repeating the primal act of creation through symbolical establishing the cosmogony based on cosmogonycal victim. This act has a purpose to "transform" the actual reality in which is the existing of the artefact-the house-the world, and to took it into spheres of higher order, implicating symbolical potential as a reality, satisfying at the same time specific system of social needs and expectations. The sacrifice has a myth function of mediator in which role it presents the centre of codificated symbolical process in which "the real thing, or its substitute, is getting to semyotical status", discovering the specific relationship of symbolical and pragmatic function of "things" in specific structure.

Through the process of substituting the building victim with cosmogonical victim, the house, presented as a human body or the body of the sacrificed animal, becomes "the base for connectioning the building victim with the anthropogonic victim". The mutual codification between the victim, the house and the conception about creation of the world, is relating the complex action of consecrating the building, with the victim-with the primal victim of ancestries or deities, so the world of man could be made, which means nature to be defeated with culture, and the chaos with cosmos...

4. 2. The threshold

In every human settlement, the threshold is placed on the passage between "inside" and "outside". The meaning of this point is structured of social and psychological connotations. This point of the house is connected with the sun-the life and its permanent re-creation. Numerous customs and beliefs which derives from the idea that the ancestries are buried in the house, and the rituals which substitutes this "burring", leads to supposition that the cult of ancestries is a base on which this complex of customs, beliefs and rituals is structured.

Respecting the threshold is notable in some examples of primal human settlements, were sometimes we can find planted trees on both sides of the threshold with a basic function -symbolically continuing the process of birth, life and death.

This part of the house lasts in time repeating in different spiritual spheres the life cycles of its inhabitants communicating through it with the objective world and the reality. Getting back across this element, means getting back to the primal darkness of the "ancient home" of all things. The act of passing through was always realised with the perfect procedure of stepping across the threshold. through the practices that leads to the symbolic unity with the ancestries.

4. 3. The hearth

As a part of complex mythopoetical structure of the house, the hearth presents that part of the object which unite the living inhabitants with their ancestries. The function of the hearth has an explanation in the great number of symbolical values deriving from its complex system of customs and rituals, and its importance as a centre of the house-the world. The supernatural specific of the fire and the hearth is a base on which exists numerous legends, stories and myths. Because of all this, and its mystical power as an archetypal feature, the hearth has the cult role in human life. Existing on that level, the hearth becomes the same as the altar in the temple, making the centre of the house, or the place where the hearth is, "zone of strength, zone of strong protection.." The hearth is a place that leads into earth as same its "internal fire" comes out of its depth upwards. Its manistic meaning is based on a fact that the funeral of the dead was made under the hearth, being on that way home and sacred altar in the same time.

As universal cosmogonical model analogue to the centre of the world in which all components of the universe are connected, the fire living in the hearth has an ambivalent function : symbolising light, knowledge, birth, sanctifying and spiritual energy, and from the other side, it represents the evil force, sinful passion, death, and the final judgement. From this derives the special respect to this part of the house, which is very important because of the fact that even today, sometimes this part is a base for the functional organisation and the final composition of the houses, although it is not clear enough : which are the connections with the traditional background, and what did really survived from this complex system today, on this level of development of technology, industry, and arts.

5.CONCLUSIONS

The technological consciousness in relation to myth consciousness, today dominate and represent the main imperative in all urban structures. It is functioning in nowdays conditions using the values of the technological civilisation. Technology, explained like the sum of the ways in which social groups provide themselves with the material objects of their civilisation, is the main point in the process of codification the axiological components in the society. Internal connections in the society, including indiviuals as same as groups and bigger communities become week and in time they brake off.

In that way, trying to get back to some archetype codes, is a serious problem bearing in mind that the technological civilisation promoted its social and economical parameters as terms that "define belonging". The result of that, is the appearing of the communities structured on different way, with common interests very different from the traditional needs in and for the community. Symbolical and irrational components of the space in that process of creating new relations, are somewhere else. They are defining some aesthetic components and values hidden from us because in the process of researching of this problems, we are working with known categories, established relations and clear positions. New methods that are produced by the new-technological civilisation are not able to help. Our interest will have to get back deep in to the past, so we can be able to make connection with the future.

The desacralization of the architectural space destroys completely the image about it as a part of a civilisationel continuity. The lack of semantic transformation of particular architectonic segments derogate the last way of interpolation of the "irrational" factor in new modalities. Thus, it is obvious that the importance of our determination for a return of the imaginary to the architecture as one of the way of introduction of one's own symbolic mythical thought, capable of discovering the most important and complex existential situations. The possibility of discovering and getting to this level, leads through the researching in the field of traditional architecture, especially through the analysis of its sacred and symbolic structure, parallel with all researching and interests about the modern architecture and the complex problems connected with it.

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HISTORY AND ARCHITECTURE

D.Kuban

Since history is a comprehensive study of the past, it includes every human endeavour. Evidently architecture is an eminent part of its subject-matter. But it is not historiography of architecture, but those artifacts themselves, subjects of historical inquiry, are the subjects of my talk. Whether we like it or not, they live with us, firstly for economical reasons, secondly for cultural reasons.

Since the modern period we thought that we destroyed history's hegemony on our life, we made a tabula rasa of the past, we re-started again. When I was at school the use of arch was blasphemy. Today we understand that it was simply bigotry. And it had been proved that it was indeed a non-sense. Then came the so-called post-modern fashion. History became 'in'. Now Post-modernism is out. But the importance of historical environment remains as it has been through centuries. Because it has nothing to do with fashion. It is part and parcel of any environment. As long as we accept a single building's importance in our life, for whichever reason, there is no need to philosophize about whether history has any significance for contemporary skyscraper. I think any perceptive observation of an historical site and its contemporary changes are more important to understand history than the volumes of contemporary pandits.

Significance of history for contemporary architecture, and more generally, the significance of history in any moment of architecture, and more so for urban environment, resides in a simple fact: synchronic existence of buildings. As I mentioned Hagia Sophia, it is the same with any historical buildings. The walls of Istanbul, or a nineteenth century house. A man passing through a street does not classify buildings according to their age. They are part of the same material continuity surrounding him or her. Artifacts. Architecture, buildings of any sort are the only visible witnesses of bygone ages, surviving, and still constituting part of our physical environment. The whole idea of conservation is a simple problem of integration. Integration of the contemporary into the past. In the pre-industrial age the passage was smooth, and the sensibilities were not offended by change. Now it is brutal, and of grand scale. So is the outcry.

Our spiritual life, our religious belief, the philosophical or traditional bases of our behaviour may be more important in the final analysis. But I think to have a shelter is as important as any belief in the life of a common man. On this observation one may create a religious philosophy of architecture. While in history buildings were not object of worship, but they were object of reverence. Mosques, Churches, temples, tombs of religious men. Although related to faith, this importance has but one step from becoming the object of religious reverence for themselves. Like the icons of orthodox religion. We may create utopias on the reverence of simple shelter, as the abode of common man. It is strange that humanity created religion out of fear, but not out of positive virtues of creativity, such as the production of food and building of shelter. Although in antiquity the fertility rites, the observance of the seasons, the festivals of spring and the like had religious significance, it seems that nothing was more important than the fear of dead.

In our everyday life, our surroundings is a synchronic enclosure which follows us in every step. Buildings are imbedded in our unconscious environmental perception. If buildings have their stories to tell, whether they are old or young. Even if we don’t know the date or style of the mosque of our quarter, and we don’t give it any attention, we know that it is there. It has a fountain in its courtyard, and a minaret at the corner out the street. People gather for prayer, or for the ceremony of the deceased. Then it is the focus of our activity which brings in crowds, hinders or stops the traffic. Then it becomes a conspicuous item of the environment. The corner coffee house. An old wooden shack may have the same relevance, in our life.

Man can ignore science, religion, philosophy, moral, politics, but he cannot ignore the house he lives in. The impact of physical space and the buildings surrounding it, that of our neighbour, or the building we work in. This immediacy has nothing to do with history. Their simple existence acts upon us. But this existence which we barely notice in our daily life, is, from the point of their shape, is diachronical. The urban environment, in most cases. Is not from the same time. This is why historical architecture in all its dimensions, is part and parcel of our contemporary life.

On these observations and preliminary remarks we can construct a new program, or a new theory of total environment, not on the premises of a future architecture, but on the modalities of change of an existing urban environment. We should not speak of history, as something already past. Because as far as the human artifacts, as architecture or cities, history is with us. Although we destroyed a great deal, it is still with us. Every moment past is history and we live on the edge of history. In fact the present is the this edge. The future is a projection. If we look at our environment in this perspective we see that every item has a history. Ten years old, thirty years old, century old etc.. Once authorities decided, concerning the classification of historical buildings, that buildings until 1900 were historical, and those later unhistorical. The absurdity of such a concept is evident. History as far the history of human production of artifacts does not stop in any moment. And each moment has its significance.

Thus if we return to our theory of total environment, we may accept that every building has its intrinsic historical value, and we can consider every design, whether on urban or architectural scale, as part of a changing and stylistically heterogeneous urbanscape. We don't impose our will. We submit to the existing situation. Every new is integrated into the existing with necessary changes. As in a crowded autobus. This should not be interpreted to be conditioned by the past. Because our buildings have also their style and their idiosyncrasies. But the point here is that we don't declassify the old. We try to harmonize with them. To harmonize with the existing does not mean to copy, or to use old details. This approach is contemporary as it goes: one can choose its own way. Only condition is that there is no time limit, there is no dichotomy of old and new.

Since the existence of the old is inevitable, we have to ask the question: Does it serve anything? Do we learn from history? Should we look at history to teach ourselves about architecture? Or should we repeat the modernist approach: History doesn't say anything as a depository of architectural wisdom. Down with history!

Historically we know that imitation (Mimesis) of historical forms has been a most ordinary practice throughout history, including our faithful post-modernists. And think about the contemporary American cities. We know by experience that by the force of sheer symbolism of a religious nature, all contemporary Turkish mosques follow, rather pathetically, classical Ottoman models. Even if, as conscientious architects, we abhor this practice, it is nevertheless there.

Yet what I intend to propose has nothing to do with this practice. We should investigate the nature and limits of influence of the traditional forms on architectural behavior, even if we flatly reject them. Here architectural behavior is understood as consciousness concerning the design process. On this investigation our starting point I the inevitability of historical forms in our environment. But let's put aside this aspect which has political, ideological, and economical incentives, and which existence we cannot negate.

But let’s consider the existence of a nice old neighborhood with exquisite wooden houses: We cannot ignore them. We could consciously refuse to imitate, or to repeat them. But even if we continue our own contemporary practice, we are aware of the existence of certain proportions of windows, or an exquisite balcony, a carefully designed garden wall, a stair, a gate. Even if unconsciously, we perceive three dimensional relations of a well designed building. We are not immune to the influence from the past?

Evidently nobody intend to build another Hagia Sophia, yet any intelligent and sensuous being, especially if she/he an architect, will be overwhelmed by the light of the great church, by the dimensions of the immense interior, and by the texture and color of the walls and columns Can we simple erase from memory such a view? And can’t we ask whether this image has improved our architectural imagination? We can repeat similar observations, in an antique site, in a baroque church, in a medieval chiostro, in a Japanese garden, in the great mosques, or watching Gaudi’s buildings, or the portals of the great mosque of Divriği. I don’t know and I don’t assume to know the nature of influence of such an experience on the imagination of an architect? Eighteenth and nineteenth centuries are full of this kind of anecdotes, and we know from our own experience that styles are born by imitation. Since we intend to comprehend the relationship of traditional environment and contemporary designer, or the relationship between the old and the new, we have to understand the limits imitation, and the limits of interpretation. Maybe because we are still under the influence of the modernism, we speak not of a direct impact of the past, but a diffuse, and vague influence of shapes, harmonies, colors, textures, relationships, masses, spaces, and their challenge to architectural imagination. This is like the Tao of Chinese philosophy. It is there, everywhere, it has no name, it has no quality, but it is the way. Total environment in its historical dimension may be considered such an unnamable experience.

There will always be a question: a number of people in the audience will ask whether this derelict, obsolete and, seemingly ugly old neighborhoods could ever challenge the sensibility of contemporary architects? By personal experience of long years and circumstances, I believe that, If not prejudiced, the eye does perceive originalities, small but delightful experiences, even if the mind is unaware. Here environmental psychology may add, if not conclusive, but conspicuous insight.

There is also a moral aspect of this discussion: As children of modernism (because I believe that, although enlarged our vision of form, and freshened the architectural discourse, post-modernism was a reaction and aberration of short life) we have a spontaneous reaction to accept past experience as of paradigmatic value. An attitude perhaps inconsciously imbedded in our architectural behavior.

Evidently we are not supposed to be the champions of the past, but rationally we cannot ignore that only the past, even that of the last half an hour, we are dealing with. The future will be designed through a reaction, or through an interpretation, but discontinuity does not exist. The language is created before we start to use it.

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A STUDY OF INFLUENCES OF BYZANTINE ARCHITECTURE ON THE OTTOMAN ARCHITECTURE

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ABSTRACT

The great influence of Byzantine Architecture on the Ottoman Architecture cannot be denied. But the Ottoman Architecture is not a continuum of Byzantine Architecture. It must be interpreted as a full synthesis of original parameters and factors. The history of architecture all over the world is full of examples of the influences exerted by one style upon the other. For example, The Ottoman Empire Soliman the Magnificent seems to be the symbol of growing dynamism and a very remarkable polyvalence. In this atmosphere, Architect Sinan finds himself among various challenges issuing from quite different origins. The Ulu Cami (Great Mosque) at Bursa, the Üç Şerefeli Mosque at Edirne are both successful developments of early Arab Mosques, that is of multi-pillared oblong, then great achievement of post conquest period in İstanbul as the Conqueror's (Fatih) Mosque or that of the Beyazıt. And also the most famous Byzantine churches: The Hagia Sophia and Sts. Sergios and Bacchus, the so-called Küçük Ayasofya (Small Hagia Sophia).

Sinan may be influenced by those important challenges, but had to create the original parameters of the Ottoman Architecture of 16th century.

1. INTRODUCTION

The Ottoman Architecture received new impulses after the conquest of the Constantinople in 1453. The Ottoman Architects enriched their architecture adopting some Byzantine architectural elements. Ottoman Architects have been very impressed by the largest dome of Byzantine Architecture, the Hagia Sophia and by the idea of using semi-domes to enlarge the internal architectural space. The climax of a synthesis of architecture as well as decorative and technical practice in Byzantine Architecture is Hagia Sophia. It was one of the most important sources of impulse for Ottoman Architects. The Hagia Sophia embodied in the personage of Emperor Justinianus, symbolizes the power of the religion on temporal affairs. The building is erected by Antemius of Tralles and Isodorus of Miletus within 5 years (Consecrated in 532 AD).

The great church of the Hagia Sophia, constructed in Constantinople (İstanbul) by Emperor Justinianus in 532-537, is unparalleled in premodern Western Architecture since the designers Antemius and Isodorus are known to have been geometer and natural scientists.

Historians of architecture often explain the Hagia Sophia's construction in terms of what might best be described as a technological design revolution.

This perspective is explained by Antemius and Isodorus as: "What their contemporaries called MECHANAPOIOI GROUNDED IN THE THEORY of statics and kinetics and well versed in mathematics…. That could be applied to the practice of either engineering or building. Be it a steam engineering or the complex vaulting of Hagia Sophia.

They were one would like to think, not architects to start with, but they turned into achitect when called upon to devise the plans and statics of a building never before considered viable on a large scale. On the other hand, historians of technology who consider Galileo. Galileo's 17th century "Dialogs Concerning to New Sciences". Where in the first "science" is strength of materials to be the seminal work of structural mechanics.

In fact the development of theoretical mechanics to appoint where it could begin to treat structural problems as complex as a vault is of relatively recent origin.

Yet if the use of structural theory for the design of the Hagia Sophia's lofty structure is denied, we are still left with the problem of explaining this important and remarkable technological achievement. Historians of architecture often explain the Hagia Sophia's construction in terms of what might best be described as a technological design revolution.

It will be better to make here a comparison of Hagia Sophia and Süleymaniye Mosque of Sinan, for explaining clearly the influence of Byzantine Architecture on the Ottoman Architecture.

2. ARCHITECTURAL AND STRUCTURAL CONCEPT OF HAGIA SOPHIA

To begin to explain the query, it is necessary first to reconstruct the original structure, since the building we see today has undergone extensive modification. From surviving sixth century accounts, we know that exceedingly large deformations of main piers disquited the Hagia Sophia's builders even before construction was completed. The central dome feel after being subjected to two earthquakes, first in December 517, and again in August 553. A nephew of Izodorus than erected the second dome having a higher profile than its prodeccor's. The form of the second dome remains basically unchanged despite its partial collapse first after an earthquake in tenth, and again after another in the 14th century.

Structural repaires associated with these incidents, and other adversities, involved the placement of much additional buttressing and thus the building exterior has been greatly altered.

The Building has a rectangular plan, the width and length of the Building are reconciled with each other. It does not create a disturbing effect despite of its enormous spatial dimension. It is accepted as the most striking example of the combimution of basilical and centralized plans in church buildings.

While the basilical form presents usual, special division, the centralized type is specified through a middle square. The middle square is covered by a ribbed dome and is lengthened by two half domes fitted between the side arches consequently having the same diameter as the main dome.

The central main dome whose span measures 31.56 m. along east-west axis and 30.50 m. on the north-south axis is supported by arches which on the east and west braced by semi-domes and huge pendentives. This enormous pendentives transfer the loads of the main dome and semi dome to the arches of the huge dimensions.

The arcades on both sides on the piers along the periphery of the apse follow the inner surface of piers.

The load transfer of the archers bordering the main space below the dome and their connection of the is ensured at the rear by means of half arch. Two lower coloumns transfer the thrust by arches into outer wall.

Hagia sophia has a magnificent interior organization. Its design in the development of the creative performance in the architecture of that

The most striking feature of this monument is balance between void and solid.Dark and light that produces a mysterious effect. The array of windows around the base of the dome of Hagia Sophia that yield a diffuse light and create the illusion of the dome suspended above vast interior space is throught to have originated solely for visual effect.

The Hagia Sophia has been planned with scientific ripidity. It is the superimposition of all pythagorean triangles, this purely intellectual principal that formed the basic idea of the architecture, contrasting or not with the purpose of the building.

The structure of Hagia Sophia can be quite clearly identified from the outside. It is marked inside as an example dematerialization throught optical effects. In no other building of the world the contrast of the form with the illumination was ventured so daringly.. the contrast of the architectural whole was never formulated so phenomenally.

The method of planning was most probably a gradual approach calculating now in past and now as awhole, But the real question to us is how for the architectural conception had or could have already been developed at the initial planning stage.

Recurred defects and disasters made it necessary that the huge building had to be restored several times. Thus for instance the four pices together with exterior buttresses maintaining the equilibrium of middle part of the facade represent at most measure for protection of the building. The vaults of the buildings was repaired in 10^{th} century after the earthquake demages.

3. ARCHITECTURE AND STRUCTURE CONCEPT OF SÜLEYMANİYE MOSQUE

When architecture was appointed chief imperial architect, he was confronted with five challenges demanding his inremitted respect and a commitment to surpass them. And after had costructed the famous mosque Süleymaniye.

The first was the famous Ulucami (Greatmosque) at Bursa (1399) which resolved the multi pillared system of traditional Arab Mosqoues, adopted by also Seljuks, into a multi-domed one.

The second keywork was the Üçşerefeli Mosque(1437-1448). At Edirne in which the architect succeeded in covering the huge rectangle using only two mighty piers.

The third was the first imperial mosque in the Capital. The Fatih Mosque (1462-1470) built soon after the conquest with rectangular prayer hall ingeriously roofed by centre dome supported on the east by a semi-dome of the same diameter and on either aisle by three small equal cupolas.

The fourth is Beyazıt Mosque (1501-1505) whose architect took the Hagia Sophia (the Byzantine monument) as a guideline. Hagia Sophia is an important challenge of Byzantine Architecture. It was costructed in (532-537).

Finally appears a challenge the famous church of S.S.Sergius and Bacchus (527-536) where early traces of the potential offered by centralized dome sitting on an octagonal basis are strongly felt.

Architect Sinan had on one hand to follow the traditional strict style on the Sultan Mosque and on the other hand to surpass Hagia Sophia.

After one millenium of Hagia Sophia construction, Sinan decided to adopt the plan of Hagia Sophia in Süleymaniye Mosque.

The main aim in Ottoman Mosques is the monumentality. Sinan achieved in a very skillfal manner the monumentality in Süleymaniye Mosque compared the Hagia Sophia.

Süleymanite Mosque has three entrances leading into the courtyard, the middle one as the Royal Entrance. The forms and solutions adopted from antique on Christian Architecture have been interpreted and applied here very individually. The stalactide arch and plasters, as well as with its projection and height can be considered as an independent architectural monument by itself.

Süleymaniye's measurement of 63x69 m. conform absolutely to the symbol of the perfect circle down in perfect media. Sinan had a great admiration of symbolism of the circle and had prefered the pure geometry.

The dome of Süleymaniye arose from his direct study of Hagia Sophia under the aspects leading to his formation with the Ottoman way of life.

The dome of Süleymaniye Mosque is 53 m high at its crown and its diameter is 26.5 m.

The dome is placed in the middle of the plan on a drum and the load of the dome are transferred by four great pententives and arches to four huge piers.

The mosque's space is positioned along the main axis of the building extented adding to the half domes each with the same diameter.

The arches along the lateral axes are embedded in semicircular walls pierced through semicircular walls pierced through several windows. These are carried in the lower part of the space by arches on powerful columns.

Thus the inner space of the mosque has been extended also towards the side, so the feeling of spaciousness is increased. In this disposition, the traces of Hagia Sophia cannot be denied.

However, Architect Sinan succeeded the making of the buttresses in his building. The structure elements could have been conciliated by the architectural forms.

In the interior is definite axial drive towards the prayer place (mihrab) is immediately counteracted by the centralized climax of the main dome.

A man entering the prayer hall, is over whelmed by spacioussness.

All spatial divisions of the interior are reflected outwards in side, by side upon another position of different arches and openings.

On the coutrary of Christian centralized architecture, Sinan legitimized.

Sinan also had given to interactable detached for corner the function of vestabules in front corner entrances while cut off from central area.

These still form a part of the aisles.

In Süleymaniye Mosques, Sinan achieved his own inheritance, in the metamorphosis of material into asolid stereometrik structure.

The characteristic motive is here the stalactite squinches at the corners of the space where the stalactite becomes a symbol for the stress transfer between the dome and column.

4. CONFRONTATION OF HAGIA SOPHIA AND SÜLEYMANİYE MOSQUE

Advantages and some important particularities of vaulted structures seem to have been initiatively discovered by mankind in the early ages of the history. Similar particularities of domes have been taken into consideration apperantly much later.The first domed components were constructed independently in various regions of the world. An extensive use in later period of the history has likely taken place in the temples in the Eastern Mediterranean and Gulf Region countries which were homelands of various civilization of different periods history.

Erection of some prestigious edifices of the modern times were witnessed all over the 16^{th} century of Ottoman empire during which extraordinary progress was observed in the building art and especially in the consruction of domed structures.

Within this framework, the name of Sinan, the great Ottoman-Turkish architect and master builder should be remembered with veneration, since he was the man in the very origin of the mentioned progress as well as in some other spectacular engineering developments of his time. It should be also remembered that İstanbul (old Constantinople) was the world center of the dome and cupola tradition from early Byzantine times. The majestic presence of Hagia Sophia naturaly influenced all building operations in his part of the ward. Sinan is reportedly known to be inspired by the structural and architectural solutions of Hagia Sophia too, especially in his early years. Later, he attained them with the Süleymaniye Mosque and finally he almost surpassed with the Selimiye Mosque at Edirne (old Adrianopolis)

It would be very superficial to consider Süleymaniye Mosque a new remodelling of Hagia Sophia. Infact the Ottoman Architecture after the conquest of Constantinople, experienced an important change. The influence of Byzantine architecture on the Ottoman architecture persisted rather in a creative organization of high level. The Hagia Sophia became an important impulse source of Ottoman architects and Sinan and more mosques were costructed after the conquest.

The confrontation of Sinan's architecture with the Byzantine Architecture brings to the conclusion that Sinan was mainly engaged in the shape.

The exterior of the mosque became also a convincing expression of its inner space.

In early Ottoman Architecture, the formal basic unit was the domed square structure.

This basic unit has been used by itself on in recurrent successive rows.

The evolution of the Ottoman Mosque took a more subtle and sophisticated path as technical knowledge of Ottoman Architects increased. The fuction of the half-dome as a structural and special element was realized.

The objective was not to discover the ideal superstructure but to create the longest uninterrupted space. The space concept of the Byzantine Architecture was however entirely different from the Ottoman Architects. Byzantine Architects claimed the infinite space, for which they were seeking to realize as reproduction of the celestial space floating down.

The array of windows around the base of the dome of the Hagia Sophia that yield a diffuse light and create the illusion of the dome suspended above the vast interior space, is thought to have originated solely for visual effect.

The Ottomans however, achieved the "OTHER WORLD", in their spaces by way of an entire clear stereometry. The "GREAT WORLD" is not the opposite of this world but it is represented ubiquitously through the plastic and stereometric space element which is free, with no direction and purposeless.

The Byzantine and Ottoman Architectures try to overcome the gravity they seek and from their own rules beyond the law of gravity.

The courses to the goal were however of different characters. The lazy patterns in Byzantine Architecture were loosened and converted into the stalactiles in the Ottoman Mosques.

The weakness of solid matter in the Ottoman Mosques which are decisive, technically removes the effect of gravity whereas the exterior of Hagia Sophia shows a solid and plain mass compared to Süleymaniye Mosque of Sinan. The architectural effect reflected on the facades of Süleymaniye Mosque takes however its form from interior.

The domed square structure dominates the shape of the whole as an imaginary primary form covering everything.

In Hagia Sophia the designing concept of the interial organization was to create a spacious light and floating space while the roof was resting on a solid sub-structure.

Instead of spherical forms arranged one upon the other and side by side as in Hagia Sophia. In Süleymaniye Mosques there is a change between the half domes and side walls which contribute to the realization of a stereometrical mass.

Another difference confronting the floating space of Hagia Sophia is a metamorphosis of round arches supported on columns with loosened capitals.

The Byzantines avoided the glare and direct light in their prayer hall, where as the prayer hall of Süleymaniye is illumated. in the ottoman structural tradition the dome had always been supported axisymmetrically or at least by mean of four inches of the same rigidity in two perpendicular directions. In addition, the columns carrying the inches had to have the same rigidity in these directions in the sub-structure of the dome of the Hagia Sophia these rules of symmetry do not exist. This is why the Ottoman architects had never used a sub-structure like that of the Hagia Sophia for their dome. Although they adopted the semi-dome as an architectural element they nevertheless improved it structurally. Intuitively they must have been aware of the fact that the semi-dome is weak against edge loading perpendicular to its middle surface. As the longitudinal section of the Hagia Sophia, the semi-domes are relatively shallow and the inch connecting the main dome with the semi-domes is extremely weak as compared with that arranged in longitudinal direction.

That is, the stiffness of the supporting system is the different in longitudinal and lateral directions a fact which leads differential settlement along the boundary of the main dome to bending moments both in main dome and in the semi-domes.

This must be the reason of failures and damages encountered at the dome of the Hagia Sophia. Although many supporting and restorative measures have been taken along the time, it could not be prevented, that the base ring of central dome of Hagia Sophia is today an ellipse having 0.55m difference between is two principal axises the plan and roofing of Süleymaniye Mosque built from 1550-to 1556 A.D, has a square plan and covered by a central dome having 26.5m diameter. Again, as distinct from the Hagia Sophia the stiffness of the form inches is carrying the central dome and rigidity of the frames supporting the inches are equal at each corner and each direction. As a result of the symmetry, the sub-structure and the dome itself have suffered not even the slightest damage, despite many earthquakes during its life of nearly half a millennium.

The comparision fields that Byzantine architecture is closely allied to the Greco-Roman tradition and much influenced by Early Christian Architecture of Western Asia and Anatolia. While the Ottoman Architecture is allied to the Far Eastern and Central Asiatic tradition and is under the influence not only of Western Asia and Anatolia but also of Mezopotamian and Sasanid Art.

Byzantine and Ottoman architecture are distinct in respect of culture and origin, and unique in accordance with geographical condition, under the influence of the old architecture of the orient.

1.It is very superficial to consider the Ottoman Architecture as remodeling of Byzantine Architecture. These are many examples in history of architecture, that one Style was influenced by the other although it is not possible to deny every style has its own-original individuality

2.The architecture of Süleymaniye Mosque is a synthesis of other sacred buildings and has a originality as Hagia Sophia

3.Süleymaniye Mosque sub-structure has the same dignity and symmetry in two perpendicular directions. In the sub-structure of the dome of Hagia Sophia these rules of symmetry do not exist.

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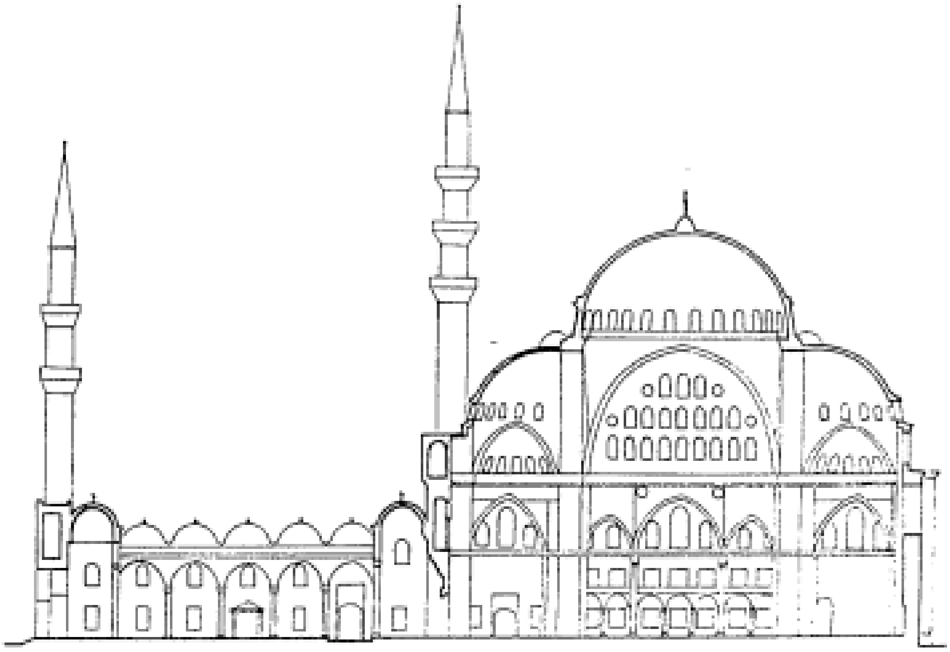

Figure 1. Süleymaniye Mosque – Cross Section

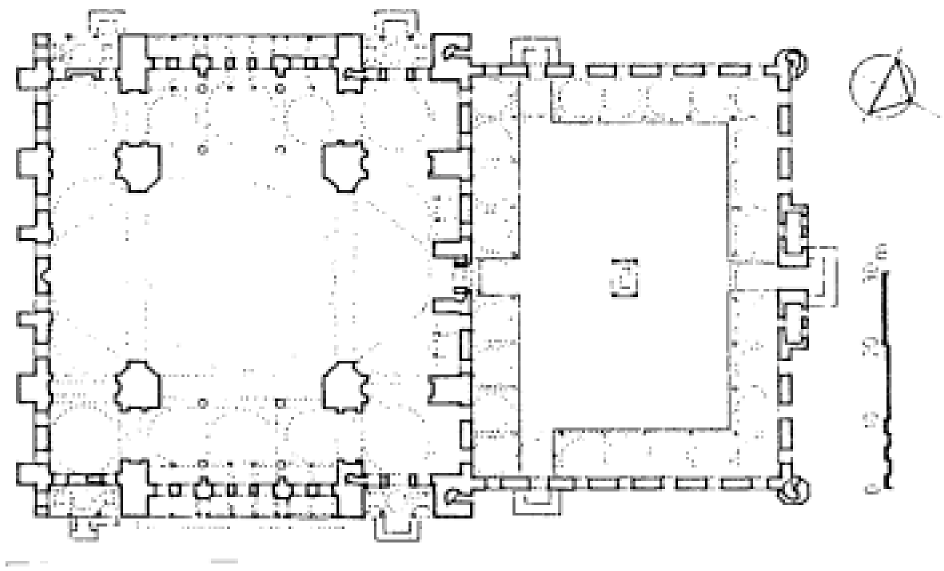

Figure 2. Süleymaniye Mosque - Plan

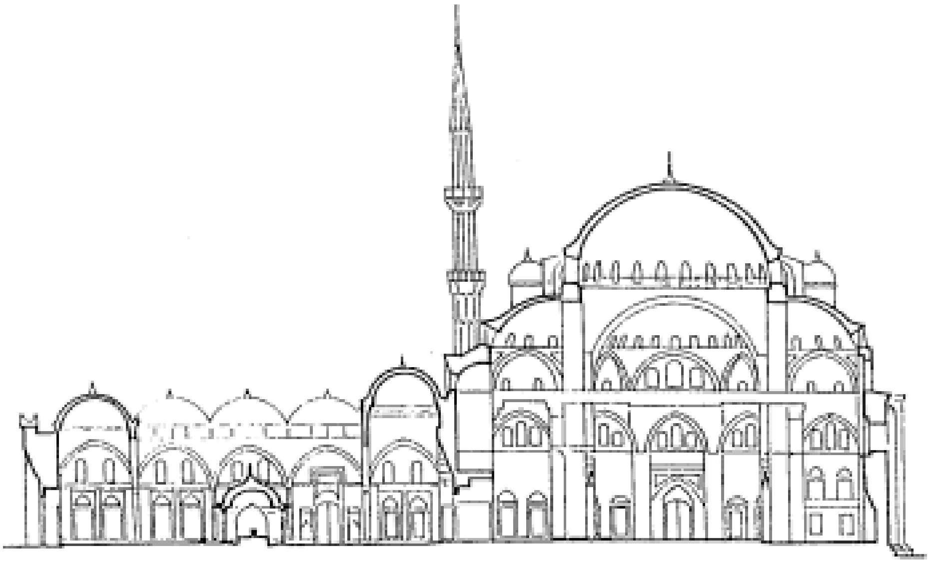

Figure 3. Şehzade Mosque – cross section

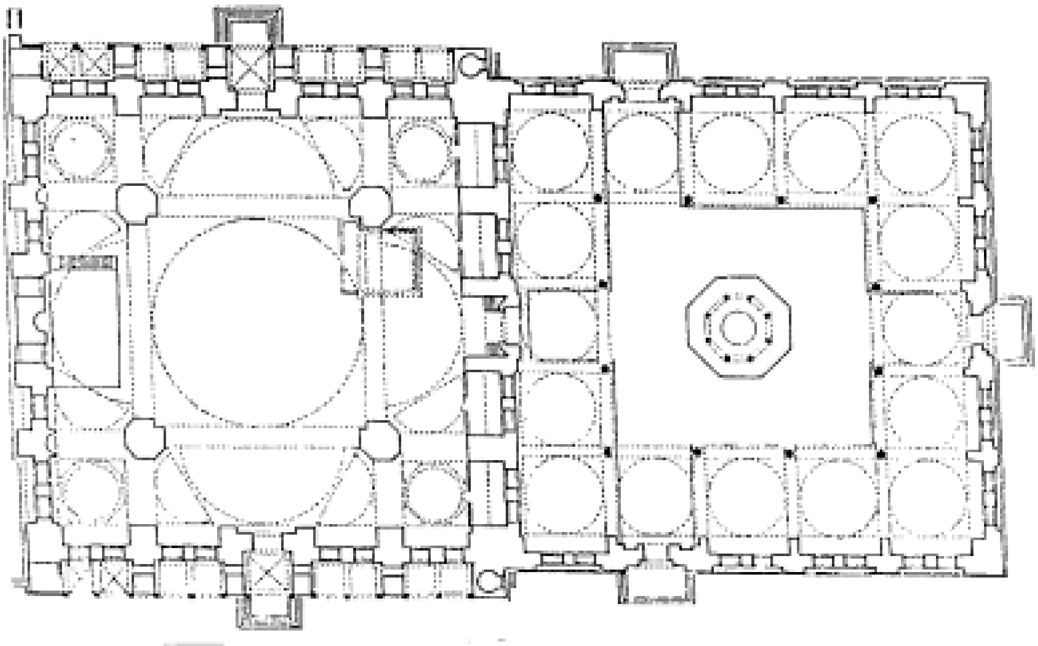

Figure 4. Şehzade Mosque - Plan

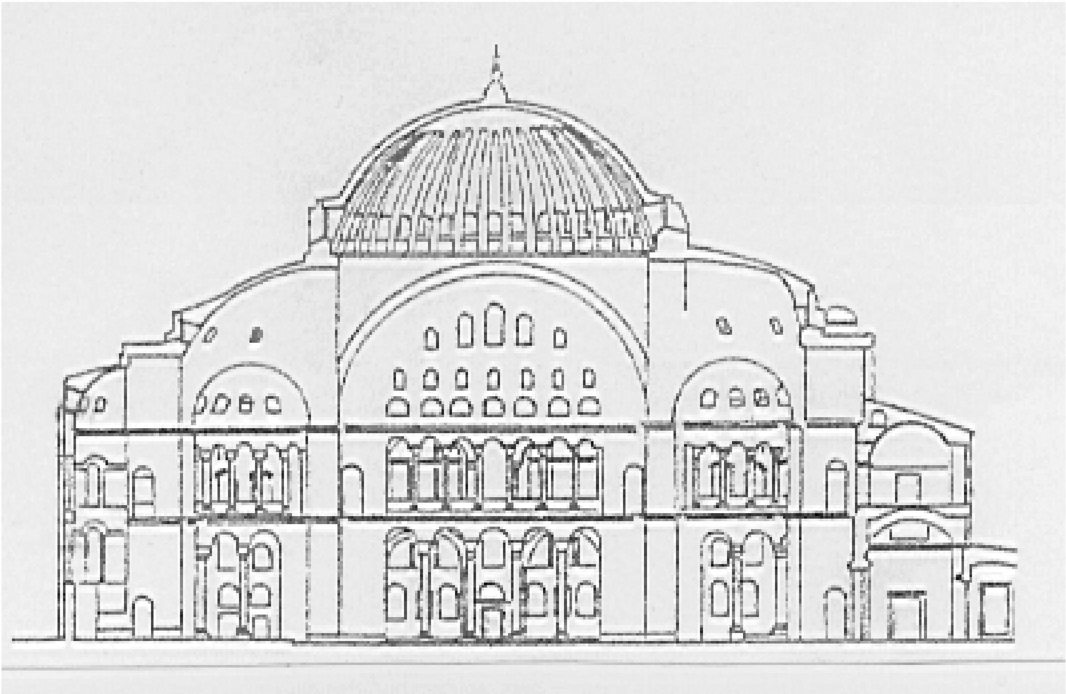

Figure 5. Hagia Sophia – Cross Section

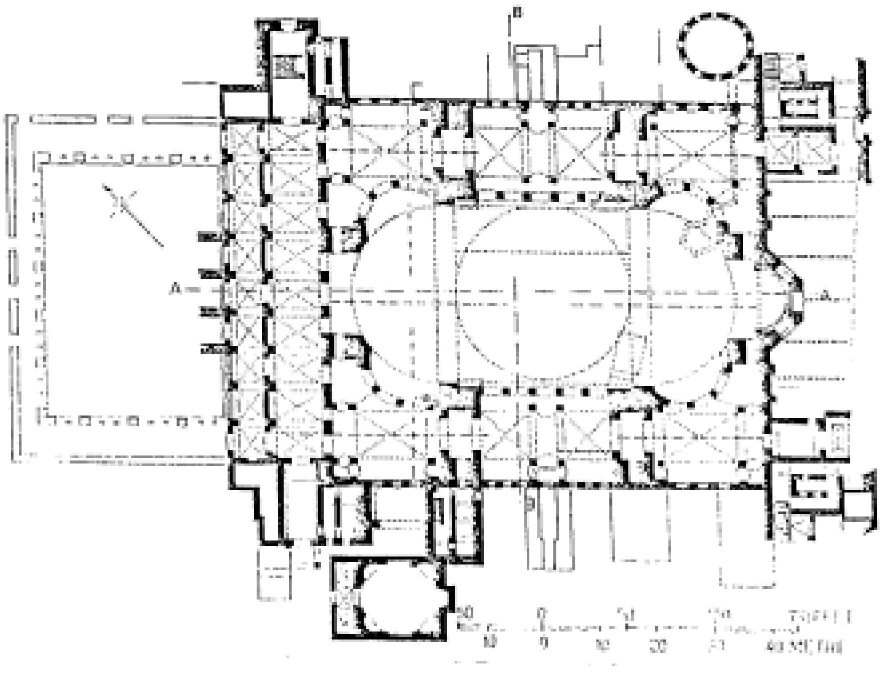

Figure 6. Hagia Sophia – Plan

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THE EVOLUTION OF EARLY OTTOMAN DOMED STRUCTURES IN EUROPE: TWO CASE-STUDIES FROM THRACE, GREECE.

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ABSTRACT

The Early Ottoman period in Thrace is represented by a number of interesting examples, where the creative blend of different cultures and influences is apparent in type, form and construction. Two domed buildings, situated on the western bank of the Evros river are presented at the paper, as typical of that period.

The first one is an open domed mausoleum (türbe) from Didymoteichon and the second an elongated barrel-vaulted han from Loutra, the Roman Traianoupolis. Both are studied in their architectural, constructional and structural parameters. The innovations they imply and the proposals they bear are established through a comparative study. Finally the structures are solved graphically and are simulated by detailed three-dimensional Finite Elements models, revealing interesting features and verifying for both buildings design competence, as well as structural sufficiency.

1. INTRODUCTION

The Greek Evros prefecture shows a rich tradition of monuments, connected directly with the two successive empires, the Byzantine and the Ottoman. The Ottoman presence is characterized by a series of early monuments, erected before the conquest of Constantinoupolis in 1453. Some of these can be considered as typical for the transformations, then taken place. We are going to examine two of them.

2. THE MAUSOLEUM OF ORUČ PAŞA

The monument under consideration is located in Didymoteichon and is known as “Tripus”, or “Pyrostia” because of its resemblance to the kitchen utensil.

The old Byzantine capital fell to the Ottoman Turks most probably in November, 1361 [2,7] and became the first imperial seat for some years before

Adrianoupolis. It had also been a favorite sultanic hunting resort, the safe of the State treasury and a remarkable pottery center, counting up to 200 ateliers according to Ewliyā Čelebi [23]. The town was also ranked as one of the most important spiritual centers of the Empire with no less than seven medresse [20].

The topography of the Ottoman Didymoteichon, Dimetoka or Dimotika differed from the Byzantine one. While the Christians continued to live inside the castle the Turks settled outside it, forming a new settlement, the Varoš, consisting (in the 17th century) [23] of 12 neighborhoods (mahalle), each one having its own mosque, (cami, or mescid) accompanied by a cemetery. The central one lied at the eastern fringe of the town and it was there that the Mausoleum of Oruč Paşa was erected [27].

According to the Sālnāme of the Vilāyet of Edirne (Adrianoupolis) of 1310 (1892/1893) [13] the building was identified with the Mausoleum of Oruč Paşa, son of the famous Kara Timurtaş Paşa [11]. Oruč himself was one of the most important personalities of the Early Ottoman period, beyler-beyı of Anatolia, with a rich building activity in Didymoteichon and the surrounding area. He died in 1426 and was buried in the city [13].

2.1. General Description

The building is a funeral monument (a türbe) of the open type, consisting of an almost square base hiding a barrel-vaulted, half-underground crypt of interior dimensions of 3.55*2.10 m^2 in plan with an entrance from outside and an upper open room of external average dimensions 5.72*5.67 m^2, formed by four stone masonry piers supporting a brick dome, not existing today, through four arches and equal in number pendentives. Strong wooden tiers connected the arches at their springing.

The lower part is built with cut porous stone applied very precisely and with a very strong mortar. The thickness of the walls of the base is about 1.80 meters. The piers have a face of very carefully cut and perpendicularly placed stone-plaques of a thickness of 20-26 cm, with a very thin layer of lime mortar between to ensure the smooth contact of the adjoining surfaces and a core of hard masonry, consisted of strong white mortar, with randomly put small and medium stones.

The arches are slightly ogive, consisted of two parts of circle with their centers a little below the level of the cornice. Their stone voussoirs are placed without, or with the minimum of mortar, while the rear is reinforced with brick masonry, bonding the arches with the dome. The dome as reconstructed by the help of the existing curves and the parallels we have in our disposal can be considered as almost hemispherical, built with a thickness of one brick.

The burial was found in the crypt, while a cenotaph must be expected according to the Desarnod engraving of 1829 and the other examples of the period on a floor upon the base. No mihrab or altar were found.

2.2. Applied metrology

The building is characterized by the application of a modulus, equal to 28,6 cm, that is 15 times the Roman and Byzantine "finger", apparent both in the general dimensions of the building and its details and basic building elements [4,9]. The bricks of the initial phase are all square with a side of 28.6 cm. The heights of the stones of the piers, the most prevailing part of the building are (in centimeters), 28.6, 49 = 1.618*28.6 = (1 + √5)/2, that is the Golden Ratio and 69 = 28.6* (√2 + 1), etc. The main part of the building, consisting of the four piers and the arches up to the cornice is forming an ideal cube of a side of 20 moduli. The same application of the specific metrology is valid for other parts of the building.

Interesting is the fact that the same metrological system with Pyrostia can be applied at the details of the Great Mosque of Didymoteichon as well as the funeral Byzantine building, found next to the cathedral church of Didymoteichon, St.Athanassios and dated back to the first half of the 14^{th} century.

2.3. Correlations and relationships

The building forms one of the first examples of the open funeral type. The ottoman funeral monuments derive from their Seljukid Anatolian predecessors, the türbe, or kümbet, of an Iranian origin [24]. This series had started about 1150 without prototypes in Asia Minor, but in its last phase after 1250 the local non-islamic architectural tradition "of Western Anatolia dominated in stylistic formulation" [3].Our türbe follows the typical form: a partially underground crypt, approached from outside, enclosed in the base and an upper prominent part [3]. The basic difference is that open type türbesi are not at all common between its predessecors and continue to be scantier than closed types, although at the beginning of the 15^{th} century we have a considerable number of them, even at Anatolia [5]. As far as their construction is concerned, brick is used, not only for the superstructure, but also for the piers, as either a plain brick-work, or a mixed masonry of alternating zones of brick and stone, the first having a close to ours example at the Omur Bey (Oruč's brother) türbe at Bursa from 1461 [6,8] , the second at the Yakup Çelebi türbe at Nikaia (Iznik) from the 14^{th} century [8,26], or the mausoleum of Ebe Hanim [6] from the second quarter of the 15^{th} century. On the contrary at Didymoteichon we have a stone masonry building.

If we search for the further morphological and constructional relationships of the Mausoleum we notice that the building comes very close to the porticos of larger buildings of the period. An outstanding example is the portico of the türbe of Yıldırım Bayezid at Bursa, re-erected after 1414 [6], where not only the structure, but even the heights of the layers of stone seem to be similar. Very near comes the portico of Yeşil Cami at Iznik, with the same alternative layers of high and low stones and even the same number of stone voussoirs (13) at arches ending at similar key-stones [8,26]. These buildings can be dated back to, or just before the beginnings of the 15^{th} century. One must also notice the similarities with the construction of Yeşil Cami at Bursa, erected by Hacı İvaz, the architect of the Great Mosque of Didymoteichon [1,8], or the portico of Bayezid Paşa Cami at Amasya, where master architect was the converted Christian Toğan Bey, also involved in the construction of the Great Mosque of Didymoteichon [8] and finally with the Great Mosque of Didymoteichon itself.

Taking under consideration all the above we can ascribe the Mausoleum, either to Hacı İvas Pasha himself, or, more possibly to some architect of his circle, probably Toğan Bey and date it just after Oruč's death in 1426.

3. THE BUILDING OF "HANA"

The second building is an elongated barrel-vaulted closed structure, known as "Hana", the han. It is positioned near the village of Loutra, the Turkish Ilıca, inside the archaeological site of the Roman and Byzantine regional capital of Traianoupolis. It is accompanied by a group of four bath (hamam) buildings, built between the end of the 15^{th} and the beginning of 17^{th} century. The place was famous during the ottoman times for its healing springs, as well as the dervish monastery upon the neighboring hill [12,23].

According to an inscription above the entrance of the main room, read by the Ottoman travellers and A.Samothrakis the building was erected by Ghāzī Evrenos, the famous general-conqueror of the Balkans [12,22,23,25]. That means it can be dated at the fourth quarter of the 14^{th} century and most probably between 1375 and 1382, when Evrenos left Komotini to be settled at Serres as its udj-begi".This makes Hana one of the earliest ottoman constructions in the Balkan peninsula.

3.1. General description

The building is consisted of two rooms. A front, small one and the rear main room with interior dimensions 10.25*8.50 and 10.20*25.30 m^2 respectively and average total exterior dimensions of 13.00*38.90 m^2**.** Two of the walls of the eastern room are preserved in a maximum height of 3,90 m, but its facade and the barrel-vaulted superstructure have fallen. Six visible today niches were used as fireplaces. The western, main room is still intact, covered by a semi-cylindrical dome. The barrel-vault is borne to the foundation by strong vertical walls with the help of four transverse arches. Wooden tiers of remarkable dimensions were tying the transverse arches at their springings. The barrel-vault and the two arches, corresponding to the end walls are built of brick, whilst the intermediate transverse arches are constructed of large, cut stone, placed almost without mortar. The exterior walls are built in the cloisonné system with single bands of bricks. The whole of the building is borne by a stone-masonry base, projecting slightly from the building itself.

At the southern part there are three huge windows, most inproper for the use and type of the building, as far as these rooms were always poorly lit [24]. Most probably the initial constructions were meant to be fireplaces. Three other small windows, pierced on the north wall were intended to throw sufficient light.

We may presume that the building was used by passengers, more probably public servants who were travelling along the Via Egnatia, the famous street of the

Roman and Byzantine times which had obtained a new importance after the Ottoman conquest .

3.2. Correlations and relationships

The barrel–vault was frequently used in Turkish building construction and especially the Seltzukid caravanserais during the blooming period of 1204-1246 [19,24,28]. The use of transverse arches was extensive as well; we find them at the 62% of the Anatolian Seljuk caravanserais [28].

In these cases the structure is not consisted of a barrel-vault alone, but of an elongated hall, accompanied by other barrel-vaulted halls, either set laterally to the main one, or parallelly to it, implying a certain static significance, in order for the central large room to be supported efficiently and in turn facilitate the terms of use. The various types of caravanserai are differentiated by the type of this lateral support. On the contrary in our case we have a rare example of asingle-aisled han with no inkeeper's room, or storerooms, or prayer hall, or private bedrooms. The fact that made the addition of the parallel vaults useless must have been, besides the possible adaptation to the desired use, the soundness of the structural efficiency.

Of course single aisled khans existed, both in the 13th century Celjukid architecture and the later Ottoman constructions of the 15th century, though very few. Nevertheless comparison can be made only with later examples, especially the Düğer Kervansarayı, built perhaps half a century later than Hana. The second part of this building, kept for the animals and carriages forms a single-storeyed, single-aisled hall with interior dimensions exactly these of Hana and diaphragm brick arches supporting the stone barrel-vault [17]. The main difference with our hall lies in the use of materials and the form of the curves which are not semi-circular but pointed. The other close parallel is according to M.Kiel the Ghazi Mihal Bey at Gölpazar, near Bilecik, again from the second decade of the 15th century [12]. On the other hand the earlier Seljukid forms vary so much in size and form that can not be correlated to our building.

Nevertheless if we use the classification Yavuz has proposed for the Anatolian barrel vaults with transverse arches, our building belongs to the first category, where the barrel-vault leans immediately upon the arch of a rectangular section and between them there is only a layer of mortar [28]. In our case arches are thicker, obviously because of their structural importance and the size of the span they bridge: At the Anatolian Seljuk vaults the arches bridge smaller spaces and the distance between themselves is also smaller, that is usually between 2,5-5 m, thus forming a square network of supporting elements, whereas in Hana there are formed squares of approximately 10,00 m.

Moreover where in Asia Minor, in both earlier and later forms the barrel-vaults consist of stone masonry, in Hana we have a brick barrel-vault on stone transverse arches. Also opposite to Hana the curves in Anatolian Seljuk and later forms, both for the barrel-vaults and the bearing arches are mostly two-centered pointed arches, while the semicircular has a limited use [28].

4. THE GRAPHIC SOLUTIONS.

Both monuments were analyzed by applying the historical grapho-static method of Derand-Blondel, as it was dilated by G.Kozuharov [16] by inscribing a normal polygon and extending its oblique sides to the floor level in order to get the thickness of the bearing walls.

In Hana the method was applied using a normal hexagon and resulted exactly in the real thicknesses of the two walls, while the thickness of the system arches-piers was inferred by the corresponding inscribed pentagon, by its extension to the level of the foundation. The use of such strongly «inclined», forms resulting in considerable thicknesses proves that the building approaches the "structures of mass", constructions, bearing the superstructure, not with the (Mathematic) form, but with an overdimensioned plethoric sub-structure. On the other hand the analysis showed that the static realization was accurate and continues the Roman and late-Byzantine treatment of domed structures.

At the Mausoleum an inscribed semi-decagon gave the outer line of the piers at the height of their capitals. That implies that the role of the tiers was just to form a diaphragm level, which raised the superstructure to the level of the arches and allowed for the total elevation of the structure and the diminution of the dimensioning of the bearing system.

5. THE ANALYSIS WITH FINITE ELEMENTS MODELS

In order to realize and justify their static and constructional efficiency, both buildings were analyzed by simulating them with finite element models, using the SAP90 program. A linear elastic model was preferred in both cases, using the step-by-step procedure in order to approach the non-linear behavior, that is, in cases of emergence of failure at critical points. The lack of tensile strength was confronted by removing the relative "transmitting" ability at these points, surfaces or their neighborhoods.

Building the model we used only 3-D elements. In both cases the simulation was set according to their structural detailing and our analysis prescriptions. At the case of Pyrostia the model was built with a total of 1264 nodes and 671 elements, while at Hana these were 2058 and 1035, respectively. The models were solved for dead load, for differential movements of their base and for strong seismic behavior, simulating that of the earthquake of Kallipolis of 1509, September, of 7,7 R.

The structural behaviour of the buildings was proved very satisfactory under dead load, giving out stresses, not superceding the 10 kg/cm2 compressive and 3-4, tensile stresses. High prices emerged only in non-bearing areas of the base. At the other hand seismic excitation as well as even small differential movements seem to raise considerably the tensile strength to unacceptable values.

Very important is the fact, that where the tensile stress from dead load is high, the construction is adequate enough, something that shows the detailed provision and the level of the knowledge of the designer of the building.

6. CONCLUSIONS.

When the Ottoman Turks conquered Thrace they found a region with no islamic culture [14], and at the same time, with a rich building and architectural tradition, connected directly with the capital of the Byzantine Empire, Constantinoupolis. In fact the Ottoman State, although the direct heir of the Anatolian Seljuks, became an Empire only when it incorporated its European territories. The colonization of the Balkan peninsula and the satisfaction of the consequent urgent needs for new public buildings that would serve the new requisitions [15] led to the quest for a monumental and simple, systematic and functional architecture. This quest brought together various influences and elements, in a new “revolutionized” architecture, where the procedure of accepting, assimilating and rejecting reached its peak at the first half of the 15th century. Thus this new architectural expression used extensively Byzantine and other non-islamic elements. [14].

Some of the forms and types that were brought from the eastern past, such as the hans and caravanserais, or the funeral buildings, the türbesi, were not known at the new lands [14]. These were enriched by local elements and characteristics, found for example in the metrology, the constructional system and details and the static thought of the Byzantine engineers, while plans and overall shape usually came from the Anatolian Turkish tradition. It is true that Christian artists, masons and architects who had lost their old patrons, the christian court and aristocracy were extensively used by the new power [15], so that people, occupied in the new building activities were not confined to one religious or ethnic group. Ottoman State used all the potential that it owned and in fact it did it very successfully, until at least the16th century [14]. This way it produced a new synthesis, melting together various and different traditions. Its products can be witnessed in many examples all over the Balkans.

The Mausoleum of Oruč Pasha can be considered as the middle-between the early Turkish türbe, already been developed during the 13th and 14th centuries at the Seljuk and early Ottoman States on one hand and the new forms of the 15th century on the other. It developed the form of the first and presented a more elegant, slender and finally self-confident construction, using devices, such as the pendentives, or the Byzantine moduli, thus adding a harmonic tone and altering the rather awkward perception of the previous ones. At the same time it elaborated elements in a daring manner, found in other forms, such as porticos to an expression of a self-sufficient system and a form which can be considered as a combination of the typical early ottoman türbe with the Byzantine small open buildings, such as monastic fountains [21]. This way we can say that it bridges the late Byzantine engineering skillfulness with the newcomers’ typology and construction.

On the other hand Hana exploits the theme of the single-aisled barrel-vaulted hall, using local Byzantine characteristics, reaching this way a point which was easier to be copied during the 15th century in similar buildings. So it seems that this building bridges the old Seljukid tradition with the later, typical Ottoman forms. Taking under consideration that there is a large time lapse between the Seljukid and our building we incline to discover at this differentiation the Byzantine contribution. This is having to do with terms of design, as the use of the semi-circle, or constructional innovations as well as statical sufficiency, provided by the local engineering adequacy.

Moreover as far as it concerns construction, two different styles, as they had been flourished in Western Anatolia are reflected upon our two buildings. The first one, a work of fine cut, polished stone, placed very carefully almost without mortar was followed at the Mausoleum of Oruč Pasha. The second style follows the Byzantine cloisonné masonry with simple ornament of brickwork and can be seen at Hana [14,15].

Both buildings step upon clear eastern motifs, unknown to the Byzantine Thracian architecture. At the same time they use the advantages of the local tradition in design, construction and statics in order to “contribute” to the creation of the new “Ottoman” art, which is surely something, much more complex than it seems to be at first glance.

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Figure 1
The Mausoleum of Oruč Paşa according to Desarnod (1829)

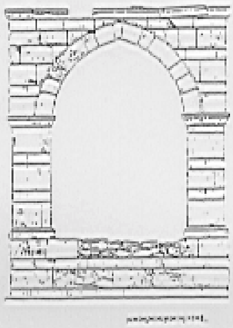

Figure 2
The Mausoleum: Drawing of the West View

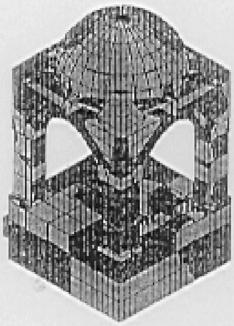

Figure 3
F.E.M. Analysis of the Mausoleum (2^{nd} step, after the removal of tiers)

Figure 4
The front view of Hana today

Figure 5
The interior of Hana

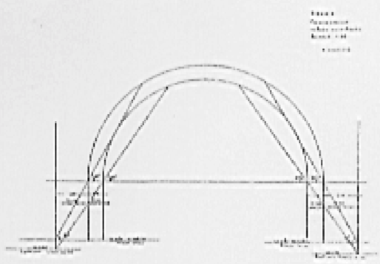

Figure 6
Graphic solution of Hana

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A "ROCK-HEWN" BUILDING IN GÜZELYURT: THE "ROCK MOSQUE" AND ITS STRUCTURAL PROBLEMS

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ABSTRACT

In this paper the rock hewn construction system and its structural problems of the Rock Mosque in Güzelyurt district in Cappadocia Region will be discussed.

No doubt, not only being a religious center makes the settlement of Güzelyurt special, but also the architectural features based on the geological and topographical existence of this region. The buildings are constructed by carving the volcanic tuff bedrocks, if needed with stone masonry additions.except the churches and dwellings, the Rock Mosque is constructed with this construction system, which is one of the rare examples in Anatolian region. The main prayer hall and primary school is constructed by curving two adjoining rock layers. The portico covered with a transverse arch profiled vault is a dry stone masonry construction.

The structural problems observed and discussed in this paper are; the problems of roof covering, the problems at the intersection part of the two different construction systems and problems caused by natural cracks .

1. HISTORY OF SETTLEMENT

Güzelyurt is a regional district center within the Province of Aksaray in southwest Cappadocia, which lies on the eastern part of the Anatolian plateau. The settlement located 1500 m above sea level, is on a porous volcanic tuff bed in the Hasan and Melendiz mountainous range; this is a grey coloured formation of 15 m thickness called *Gelveri ignimbrit* and includes an 80 cm thick layer of pumice blocks and old andesite lava pieces [1].

The eastern part of the settlement is located on another andesite layer of various phases called *Gelveri lava* [2]. This geological formation, in accordance with the general character of the Cappadocian plain and unified with Güzelyurt's irregular topography, creates a unique set of natural visual characteristics with the settlement's organic street pattern, prismatically shaped stone masonry buildings, tuff hills housing rock-hewn chapels and mosques, rock-hewn churches, passages, stone bridges, street fountains, wash basins (*yunak*) and baking ovens, all decorated with stone corbels, projections and water sprouts [3].

Historically speaking, the letters of Gregorios Nazianzos (A. D. 330-390), who was the founder of the Greek Orthodox church in Gelveri, are the earliest written documents about the settlement in Güzelyurt, which was called Karbala in these documents. In the early Christian era, Güzelyurt was and important religious settlement with rock-hewn churches, monasteries, underground dwellings and secret itineraries. [4]

The increasing Turkish threat during the 11th century and the strengthening Seljukid rule during the 12th century in the Cappadocian region caused a decline in the Christian religious activity. Following the Period of Turkish Principalities, Aksaray became a part of the Ottoman reign in 1470. In the written documents from the Karamanid and Ottoman periods, the settlement was named Gelveri. In the classical Ottoman era and until the 19th century, Gelveri was only mentioned in the *vaqfiya* documents concerning the changes in ownership. [5]

Detailed descriptions of Gelveri's economic and social life later in the 19th century may be found in the books and journals of various European travelers. Patriarch Kyrillos, who came to Gelveri in 1815, wrote that there were about 100 dwellings and 100 churches built in the rocks; most of the inhabitants appeared to be Christians, whereas Muslims formed only a minority of the population [6]. Ainsworth, who came to Gelveri in 1839, described the dwellings as "semi-subterranean"; in winter, the people "lived in caves, which were mostly built up in the front and occupied not only the slopes of the hills but also the face of the precipice to its very top and stretched up a narrow ravine that was choked with these semi-subterranean dwellings towards is upper part" [7]. In 1890, Ramsay described the settlement as extending from the narrow valley to the upper parts of the hills and consisting of rock-hewn dwellings and chruches [8].

Following the Tanzimat in 1839, the construction of privately-owned buildings was encouraged with the Land Ownership Act (*Arazi Kanunu*) of 1858. In this period, some of the Christians extended their rock-hewn dwellings with masonry additions and built a new neighborhood called *Yukarı Mahalle* in the upper part of the old settlement. The reasons why these masonry buildings were mostly owned by the Christian population were their better income and the fact that the Muslim inhabitants were still in minority. The socio-economical condition changed following the immigrant exchange agreement of 1923. The population of Gelveri decreased due to the migration of the Greek Christians to a new settlement called Nea Karvala (New Karbala) near Kavala. Their abondoned houses in the upper settlement were mostly bought and inhabited by the Turks, and the old rock-hewn dwellings in the lower settlement were reduced to store houses, barns and animal shelters. The name of the settlement was changed to Güzelyurt in 1961 [9], and it became a regional district center in 1989. Its population reaches 4,000 today.

2. THE ROCK MOSQUE AND ITS SPATIAL AND STRUCTURAL CHARACTERISTICS

Because of its importance as an early Christian settlement, there are a great number of rock-hewn chruches in and around Güzelyurt. There also is a rock-hewn mosque in the settlement, which owes is importance partly to being one of the two examples known in Anatolia. Built in the later upper settlement called *Yukarı Mahalle*, the mosque is located on top of the slope looking down on the earlier settlement; it is constructed by cutting into a solid block of bedrock situated 25-30 m above the valley. The mosque was constructed in order to serve the religious and educational needs of this upper settlement.

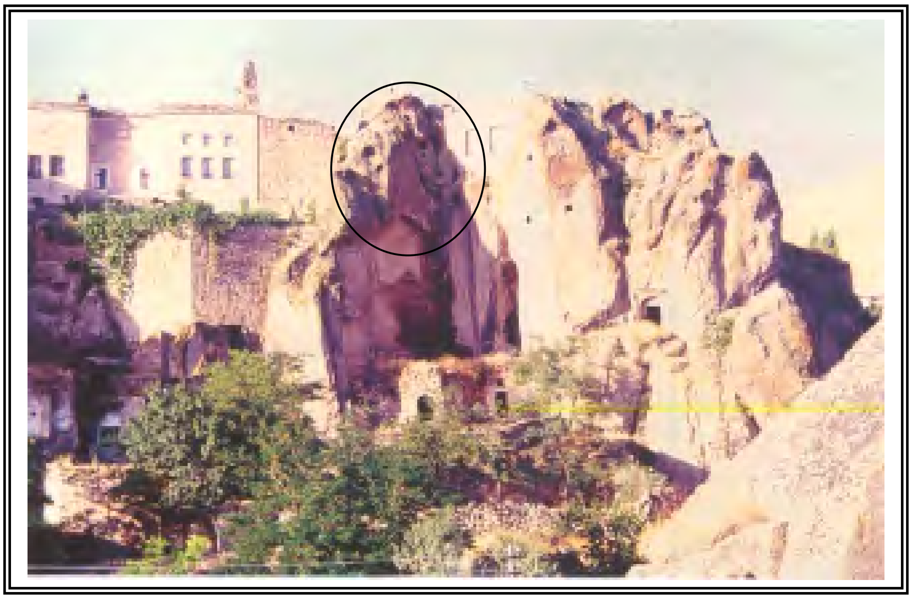

Figure 1 : General view of “Kaya Cami - Rock Mosque” from southwest.

A structural analysis of the buildings in Güzelyurt yields three different types of structural systems: rock-hewn, rock-hewn+masonry and masonry. Due to the natural characteristics of the volcanic tuff formation in the area, it was easy to shape the buildings on the slopes. The general characteristics of the rock-hewn structures were an extemporaneous geometry and organic forms. Because the main building material was the existing monolithic bedrock, the builders carved different structural elements without great concern for structural safety. [10] If it was necessary, masonry sections were added in front of these rock-hewn spaces. On flat land, buildings were constructed with an ashlar masonry of tuff blocks.

Due to its location, rock cutting and masonry systems were used in combination for the construciton of the Rock Mosque. The main prayer hall of the mosque and the primary school on the southwest were constructed by carving the monolithic bedrock on top of the slope. The additional masonry part is the portico, which was built on the flat area on the northwest side of the mosque. A flat place on top of the rock appears to be suitable for giving the call for prayer. The small hole with a diameter of 100 cm and a depth of 180 cm on top of the primary school may have been used for collecting rain water from the roof.

Figure 2 : Northeast facade of “Kaya Cami - Rock Mosque”.

Figure 3 : Portico is covered with transverse arch profiled vault.

Placed at the entrance of the mosque is a semi–open portico adjoining the bedrock, which is constructed with rough dry wall masonry technique and covered with a transverse arch profiled vault [11]. A small *mihrab* is carved on the southwestern wall of this space, which measures 4.16 x 2.75 m and has a floor of compacted earth. The portico opens to the main prayer hall through a doorway. This main hall, measuring 5.50 x 6.25 m is cut into the rock. Its approximately 3.40 m high ceiling is supported by a carved column, located at the corner of a platform next to the *minbar*. There are three carved niches of 94 cm height on the *mihrab* wall; one of them is on the left, the other two are on the right side of the *mihrab* niche. The *mihrab* is 2 m high and carved as a sequence of deepening decorative layers. On the ceiling over the *mihrab*, there is a semi–circular decorative panel. The *minbar* is located on the left side of the *mihrab* niche and is formed of six carved steps. The four rectangularly shaped symmetrical holes on the wall next to the *minbar* may be interpreted as evidence for the existence of a former wooden covering. There is a 22 cm high platform on the right side of the *minbar* and an opening to the school room. The illumination and ventilation of the main prayer hall is provided through the holes on the walls and the ceiling. One of these is placed on the ceiling near the small niche on the *mihrab* wall, whereas another one 150 cm in diameter is located on the ceiling next to the common wall with the primary school.

Figure 4 : Prayer niche (mihrab) wall of the main prayer hall.

The entrance to the primary school is on the southeast façade of the building. Altough there is an opening from the prayer hall to the school room, its smallness indicates that it served only visual and auditory functions rather than providing a passage between the two spaces. The primary school is also carved from the monolithic rock and measures 5 x 3 m. Because of the natural form of the rock, these two spaces are located almost at right angles. There is a 25-30 cm high narrow bench on the southeast wall of the school. The small niches must have been carved on the interior walls for different purposes, and there also are various openings for illumination and ventilation. One of these is a big hole over the entrance, and there are other smaller ones on the *mihrab* wall, providing vistas towards the valley.

Figure 5 : Pulpit (minber) and rock curved column in main prayer

Figure 6 : Curved vindow openings in sıbyan mektebi (primary school).

3. STRUCTURAL PROBLEMS AND PROPOSALS FOR CONSERVATION

The problems of the Rock Mosque, built by rock-cutting in volcanic tuff bedrock, show differences from the structural problems of other buildings constructed with traditional methods and materials.

One of the main structural problems of the Rock Mosque is the superstructure of the stone masonry portico. The problem is caused by the collapse of five courses of stone on either side of the centeral axis of the transverse arch profiled vault. The thickness of the insulating compacted earth layer over the vault was reduced by rain washing due to the loss of the parapet stones surrounding it.

The preventive measures directed at the consolidation and conservation of the portico are as follows:

- The consolidation of the transverse arch profiled vault through the replacement of the collapsed stone coursings.
- Replacement of the parapet stones, which supports the insulating compacted earth layer.
- The replacement of this compacted layer, which is an example of the traditional form of roof covering in the region.

Another problem may be seen on the intersection of the vaulted masonry portico with the rock-hewn prayer hall. The non-existence of a binding material between these two different structures has caused their erosion and seperation especially at the roof level due to the effect of the water sipping through the interface. The solution of these problems is through binding these two different construction systems and water-proofing them during the consolidation of the roof covering.

The main prayer hall is cut into a monolithical block of tuff. The problems observed on such a tuff formation include the following: The climatic effects of the natural conditions such as wind, rain and frost cause the erosion, cracking, breaking and seperation of the rock surface in time. Rain and melting snow water forms deep cracks due to the severe erosion of the soft bedrock; the water sipping through these cracks enlarges them with the mechanical stresses created as it expands freezing in winter. It is also known that there are natural cracks in the tuff bedrock due to its geological formation, and these cracks enlarge when the bedrock is carved [12]. The main problem observed in the prayer hall of the Rock Mosque is due to cracks in the rock formation. One of these cracks starts from the wall on the left side of the *minbar* and follows the ceiling and the *minbar* steps to the floor. Another crack is located at the intersection of the carved column and the ceiling and reaches the first one at this point. As discussed earlier, these cracks in the rock structure are formed as a result of the dead weight of the bedrock above, which exerts pressure on the roof of the carved space below, and the effects of the climatic conditions and water sippage enlarges their width; water leaks through these seperated cracks at the present. The main prayer hall is situated on a series of stepping bedrock reaching the valley bottom. This natural support limits the seperation of the cracks for the time-being.

Nevertheless in 1999, a dwelling with similar form and structure located on the other side of the valley cracked into two pieces, and the dislocated part rolled down to the valley because its cracking was not controlled. A very similar problem may be observed in Sivişli Church, which is situated at the end of the same valley; its severe condition requires urgent intervention.

Although the interventions necessary for the prevention of such problems in buildings constructed with traditional methods and materials are widely known, there is no detailed research directed specifically at such natural formations. The fact that similar problems have caused irreversable damage in other cases accentuates the urgency of implementing the necessary preventive measures in Rock Mosque before its condition gets any worse.

The analytical survey of the Rock Mosque was carried out by the members of the Yildiz Technical University, Faculty of Architecture, Department of Restoration, and the related research and conservation work is still in progress. The implementation of the necessary preventive measures must conserve and accentuate the natural formation and the visual characteristics of the existing spaces, which may provide a example and methodology for interdisciplinary research for the conservation and presentation of rock-hewn heritage that presents an architecture beautifully unified with its natural setting.

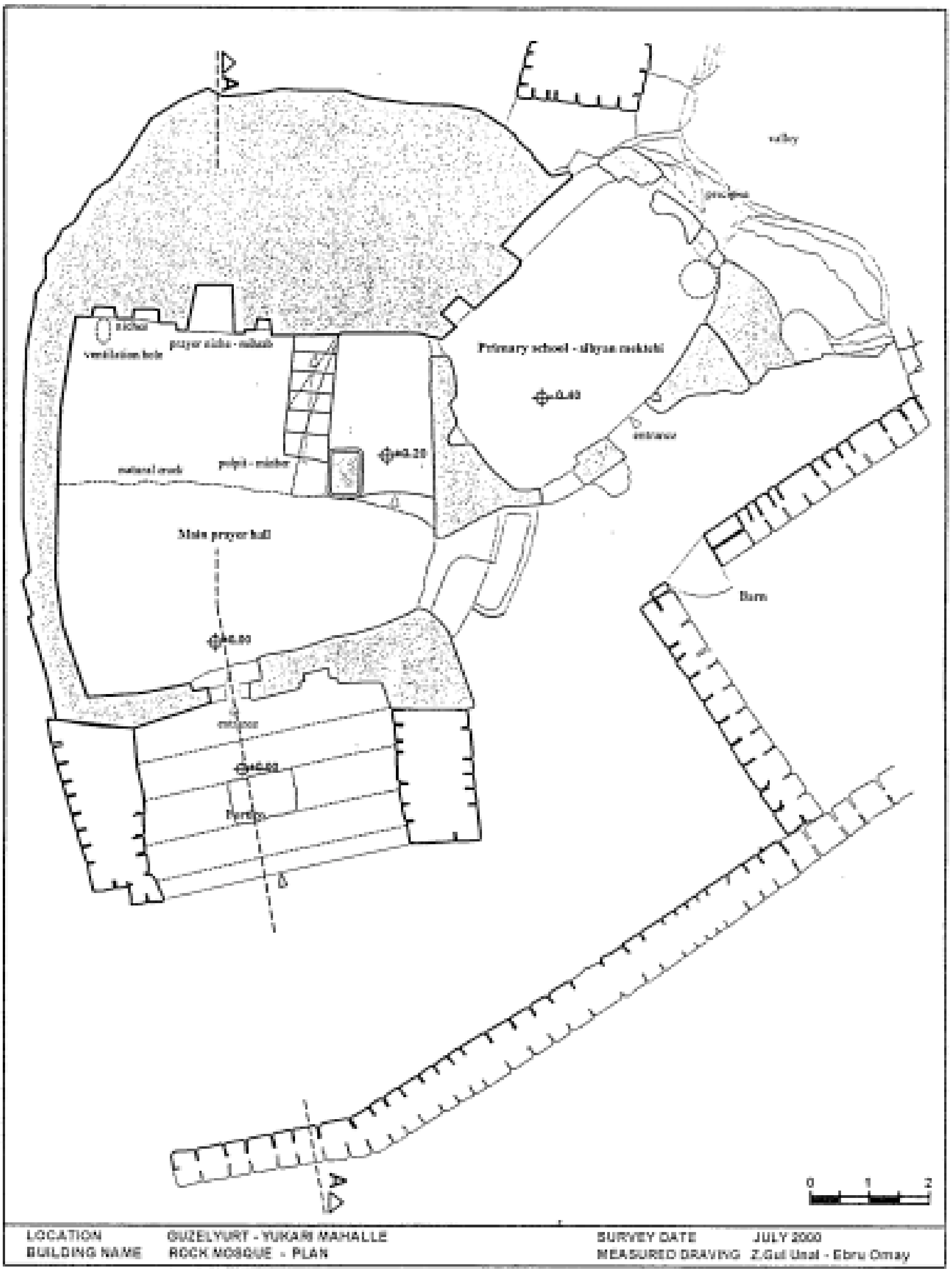

Figure 7: Plan of the Rock Mosque and the Primary School

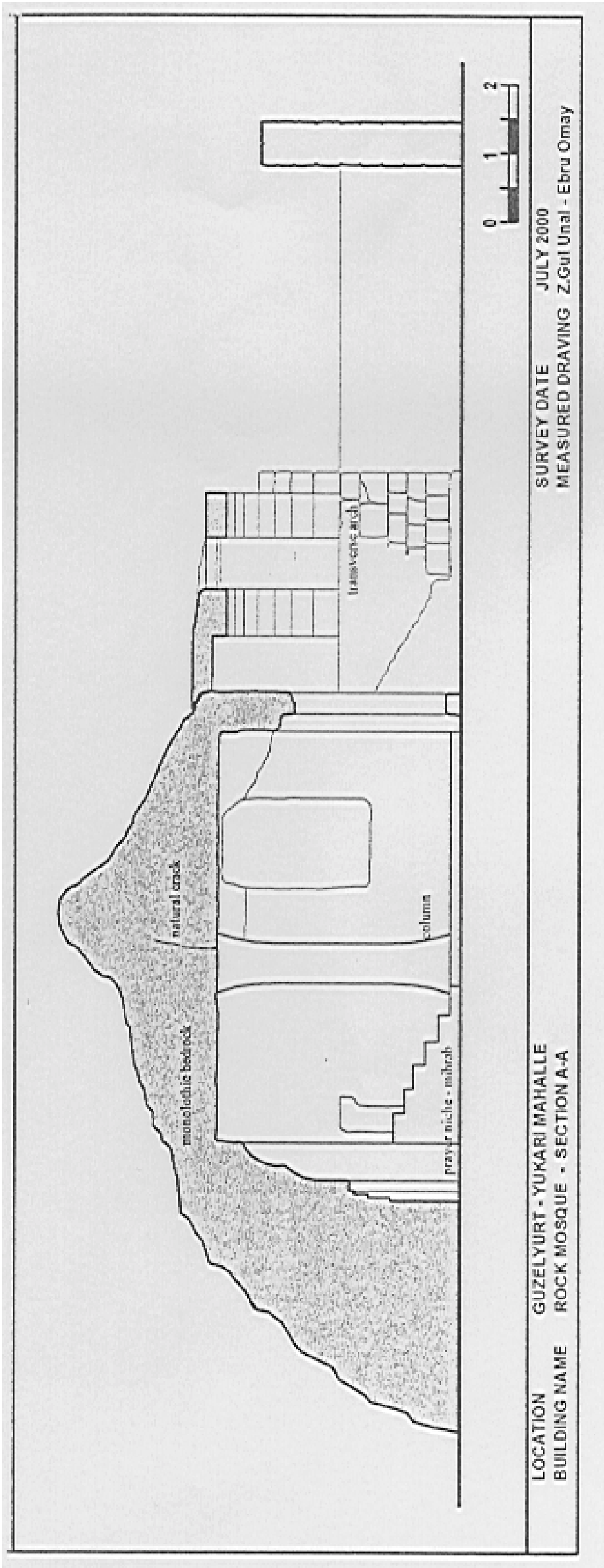

Figure 8: Section A- A , construction system of the Rock Mosque

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SPATIAL COMPOSITION OF THE TRADITIONAL ARCHITECTURE IN CONSIDERATION OF "TRANSPARENCY" AND "OPACITY"

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ABSTRACT

In this study, we define "transparency" as a component to take in sunlight like plain glass, glass block and so on in the architectural space and define "opacity" as a component not to take in sunlight like brick, concrete, wood, stone and so on. Then we use the mathematical definition to describe the perception in the architectural space by computer as a new interpretation.

In this paper, with particular attention to the history and the culture of the age, we presume various space compositions by the relation of "transparency" and "opacity". They are used in opposition from the viewpoint of multiplier effect and changeably in the different architectural spaces starting from the typical large structures like pyramid and dome to structures for religion like churches, mosques and temples.

By investigating the transition of the characteristics of the space in the traditional architecture and considering the compositions of "opacity" and "transparency" in the space, we have confirmed some kinds of steps to adopt "transparency" and "opacity", opened window on wall with short intervals in early Church style, floating ceiling by large window with long depth in Gothic architecture, dynamic composition with long and short depths in Islamic architecture and so on.

We have confirmed how people have designed the emphasis of light as an idea and how the style of light composition has changed in each era and the uses of light has spread in traditional architecture under many restrictions.

1. INTRODUCTION

This paper is the first part of four papers about "sight-depth".

Like Literature and History, Architecture has a long history to characterize each era. The Pyramid is one of the structures in Architecture that represents the dignity in geometry. Filippo Brunelleschi expresses the feelings of human beings by making use of the global sense in the laws of perspective. The objective of this research is to find out new aspect of evaluation of Architectural Space by establishing the system in which "sight-depth" has been developed. We have been carrying out the research on "sight-depth".

Traditional structures in Japan and in the countries in Southeast Asia emphasize the roof and floor. In western countries, organized method of building structures was used. In modern architecture, the interior space, the ceiling which has the difference in the height, the walls that determine the size of the interior space, the windows which give visual changes are introduced based on the organized method of building structures. Human beings live by putting their feet on the inside floor of a structure. They measures the distances in all directions, by using their eyes and memorize each image in the structure to understand the composition of the inside space in the structure.

We have been analyzing the composition of visual space for the planning of building houses as samples of using the structures which represent modern architecture such as Savoy House, Barcelona Pavilion and House Falling Water. We carried out the research on Japanese tearooms. Our research on tearooms was to have a visual evaluation of space enclosure for a tea-host and a principal-guest regarding the arrangement of the components placed in the tearooms. While carrying out the research, we tried to find out how people have been creating the living space since the ancient times, considering the change of space composition which has been changing era after era.

In the research, in modern architecture, we treated the components such as pillars walls or open windows, as components and tried to find out the relationship of each in order to confirm the new space compositions. We had never seen this in the history of architecture. In the research on tearooms, we confirmed the development of the space composition in visual space indirectly for the relationship between the tea-host and the principle guest in the flow of the history.

In this study, we define “transparency” as a component to take in sunlight like plain glass, glass block and so on in the architectural space and define “opacity” as a component not to take in sunlight like brick, concrete, wood, stone and so on. Then we use the mathematical definition to describe the perception in the architectural space by computer as a new interpretation.

In this paper, with particular attention to the history and the culture of the age, we presume various space compositions by the relation of “transparency” and “opacity”. They are used in opposition from the viewpoint of multiplier effect and changeably in the different architectural spaces starting from the typical large structures like pyramid and dome to structures for religion like churches, mosques and temples.

By investigating the transition of the characteristics of the space in the traditional architecture and considering the compositions of “opacity” and “transparency” in the space, we try to confirm some kinds of steps to adopt “transparency” and “opacity”, opened window on wall with short intervals in early Church style, floating ceiling by large window with long depth in Gothic architecture, dynamic composition with long and short depths in Islamic architecture and so on.

We confirm how people have designed the emphasis of light as an idea and how the style of light composition has changed in each era and the uses of light has spread in traditional architecture under many restrictions.

2. THE CONCEPT OF SIGHT-DEPTH

In this study we define “sight-depth” as a measure of the distance of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans. Then we use the mathematical definition to describe the perception in the architectural space in computer.

A point of view P is fixed on the speck in the space. A horizontal angle TH takes the value which continued with -PI<TH<PI about D (P, TH, PH). A vertical angle PH takes the value that continued with PI/2<PH<PI/2 in the same way (Figure 1).

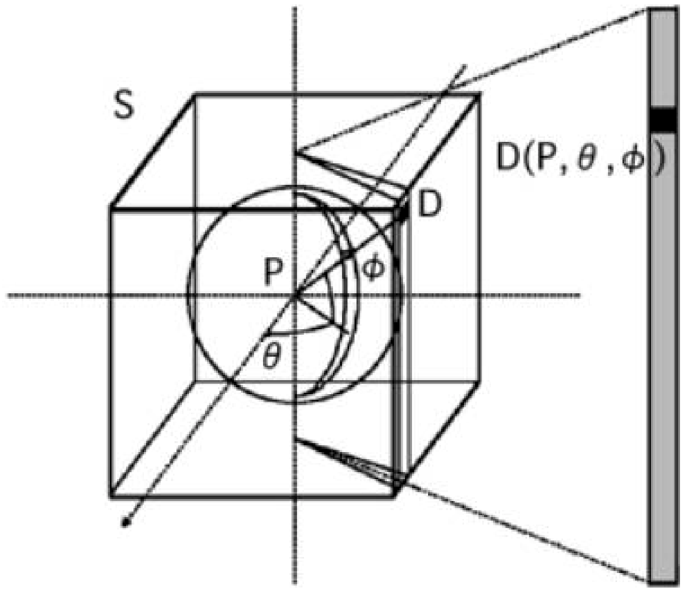

Figure 1: The concept of sight-depth

3. THE SPACE-DESCRIPTION OF THREE-DIMENTIONAL SPACE

To describe the three-dimensional information into two dimensions, we need to use several drawings for the necessary purposes of research. By the drawings, we can not show all the precise distances, directions, or three dimensional angles in a three-dimensional space. In this research, the main three drawings, which are cylinder-drawing, cone-drawing, and sine-drawing are used to describe each element precisely based on the map projection method.

3.1. Cylinder-drawing

A section in the horizontal direction is a TH shaft, and a section in the vertical direction is a PH shaft around the point of view. And, a point of intersection with the TH shaft and the PH shaft is the direction O (TH=0, PH=0) of his eyes. Cylinder-drawing is the basis of the cone drawing and the sine drawing (Figure 2).

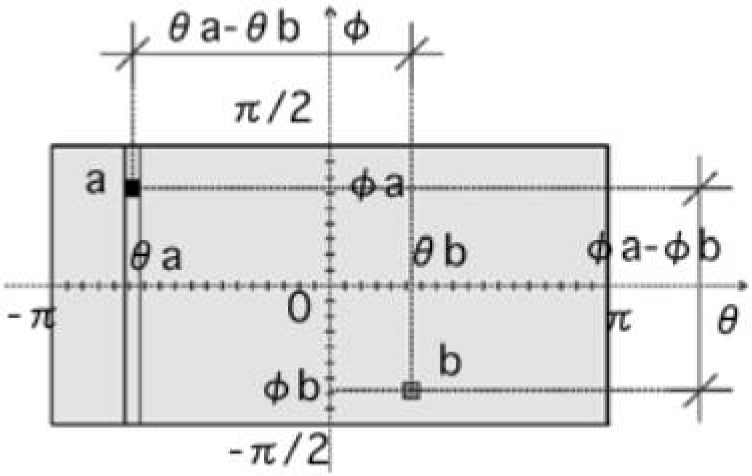

Figure 2: Cylinder-drawing

3.2. Cone-drawing

Deciding his eye direction in addition to the point of view can draw the cone drawing, and an expected angle in the optional point direction is expressed from his eye direction (the center of the drawing) (Figure 3).

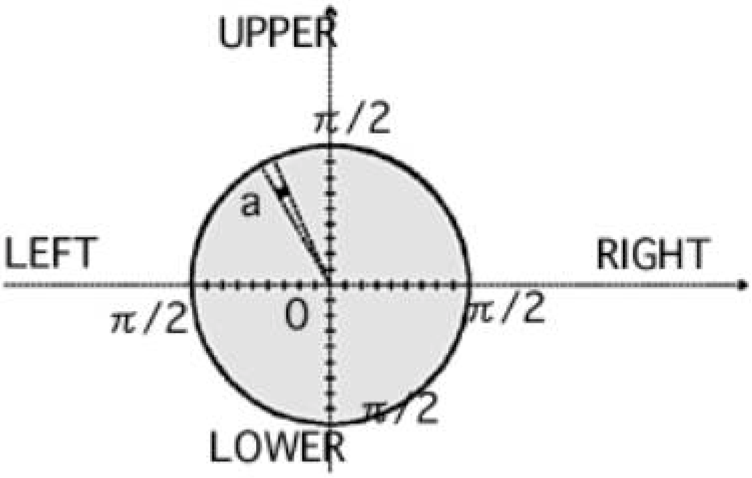

Figure 3: Cone-drawing

3.3. Sine-drawing

PH coordinate was substituted for the sine curve in the TH shaft and the PH shaft of the cylinder drawing. A rate on the corner of the solid to the applicable thing is expressed as a square. Because it warps, only square is effective in the form of space (Figure 4).

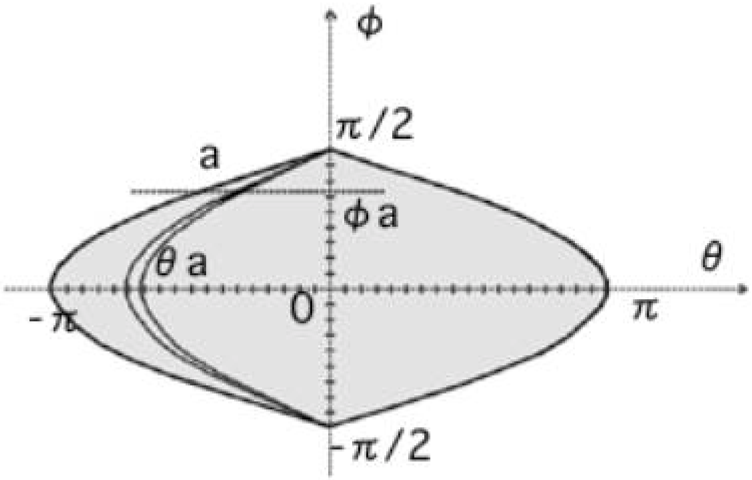

Figure 4: Sine-drawing

3.4. Description of sight-depth

Maximum value becomes 80 percents of black, and minimum value is 20 percents of black as for the shade of black and white in the sight-space description. Shade between that is expressed in proportion to the measurement value of sight-depth (Figure 5).

Figure 5: Description of sight-depth

4. COMPARATIVE OBSERVATION OF 12 BASIC SPACES

We made three types of space-descriptions which we defined in this report to have comparative observations of the methods of space-description. By defining the 12 basic eye positions from A to L, for the various arrangements of ceiling, floor, walls, and pillars. When we had analyzed each detail of the space-description, we confirmed that there are various spaces describing the arrangements of ceilings, walls, and windows.

At the connecting part of F, the wall which works as the connecting part creates two large spaces which divide the entire space in half.

Cylinder-drawings is described with PI cycle. The difference of lightness and darkness is very little near the eye point. Four main pillars G are arranged with the sense of very good rhythm. Compared to the connecting part F, the sense of existence of the four main pillars is small.

It is described that high ceiling L, has twice as high as A, three-dimensional object because of the darkness that is concentrated on a concentric circle. Since the distance to the ceiling is long, the darkness is concentrated in the upper part.

5. DEFINE OF “TRANSPARENCY” AND “OPACITY”

In this study, we define “transparency” as a component to take in sunlight like plain glass, glass block and so on in the architectural space. Transparent object divides the interior design and the exterior, and divides the space in the building. Especially, the wall is far more important, and often started from the arrangement plan of the wall by the architect in an architectural design in deciding the capacity of the space.

We define “opacity” as a component not to take in sunlight like brick, concrete, wood, stone and so on. The wall, the roof, the floor, and the ceiling and so on divide the space. The factor concerning the development of the lighting is the glass as the opacity material of light as well as the structure. The glass is an extremely old material. It had already been used as a window material in Romanic etc.

6. OBSERVATION OF 52 TRADITIONAL ARCHITECTURE

The method named clerestory of the lighting was adopted in the shrine architecture of Egypt. A central hall of the multi pillar type is supported with a high column, and the pillar in the hall on both sides is low. Rectangular high windows were made in the height difference.

The window has not developed because the house was a courtyard form in ancient Greece.

It was not rare that a round skylight was opened to uppermost in the dome part like the pantheon in Rome. Moreover, the technique of catching light from the side of vault was often used. The technique of clerestory has already been used in the church of Early Christian style. And, it caused religious atmosphere.
The arch window of the Rome style was succeeded to the Byzantine style, the Islam style, and Romanesque. The method of fixing the glass into the marble frame was developed in a dynamic space in Hagia Sophia of Byzantine Empire.

The window accomplishes large-scale transfiguration in a Gothic age. A reasonable structural method of Gothic architecture has rapidly increased the area of the window.

In the Renaissance style, the window recurs simply of the Romanic style as a reactionary to Gothicism.

In the baroque, the curve and waving respect are remarkable. The window also multiuse the oval and the curve, and various expressions are given to the interior design.

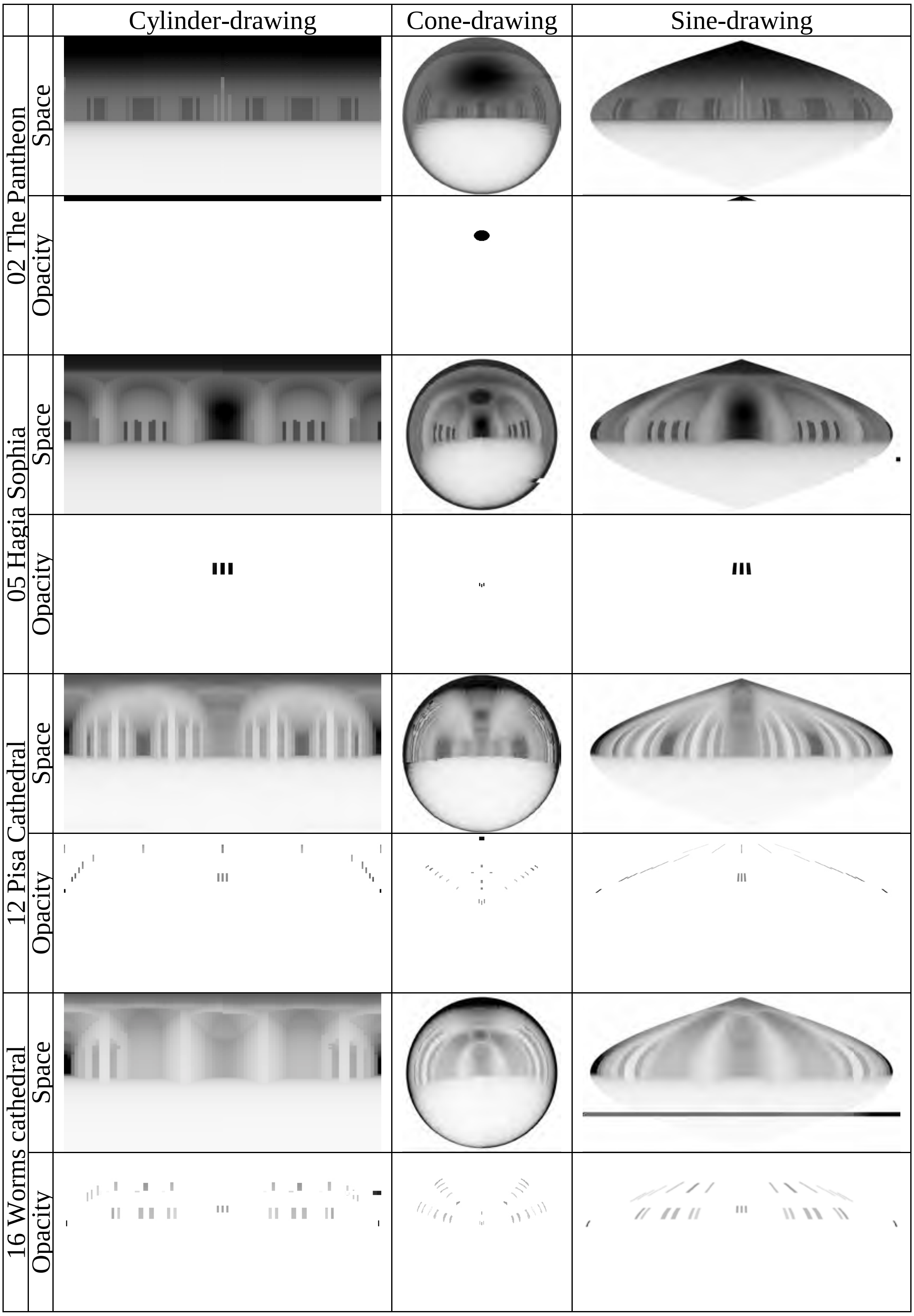

Figure 6: Example of space description of traditional architecture

7. CONCLUSIONS

The origin of the window is old, and almost the same as the start of architecture. In the history of the development of the historical architecture of the piling type structure, the opening was not structurally installed easily. The history of the wall processes the load, and is continuous of the device of securing of the lighting.

The lighting of the temple construction of ancient Egypt was done with the high window which had been installed in the upper part of the colonnade. The lattice of the stone has adjusted an incidence amount of the sunlight.

The arch has developed in Romanic. An epoch-making technology which distributed the load right and left contributed to the expansion of the opening of construction.

The development of an architectural technology from Romanesque to the Gothic is development of the technology of the space composition with vault. A high window of the cathedral at a Gothic period is composed by a beautiful stained glass. It became possible by a structural technology of the space , for example, cusp rib vault and flying buttress.

Steel and concrete have developed in the architecture at modern ages.

They expanded the range of a structural possibility before modern ages.

The space composition of architecture with an overall window of the curtain wall construction with the glass etc. appears today.

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SPATIAL COMPOSITION OF JAPANESE TEAROOM IN CONSIDERATION OF "TRANSPARENCY" AND "OPACITY"

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ABSTRACT

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In this paper, with particular attention to the history and the culture of the age, we evaluate various space compositions by the relation of "transparency" and "opacity" of different architectural spaces starting from the basic tearooms at the dawn in the 15th century to the various tearooms after 16th century. In the enclosure, the visible space that exists as a human environment changes effectively by sunlight inside the space.

We also research objects include the position and the form of the elements, opening, partition ceiling and so on. The change in the visible space caused by the movement of partitions. The effect of the changes in the standpoint and the eye direction is found to be remarkable.

We have confirmed what spaces in the tearoom that compose pecking order between the host and the main guest that needed to be maintained to represent their human relations, how architects of tearoom have designed the emphasis of light as an idea and how the style of light composition has changed in each school and the uses of light has spread in tearoom.

1. INTRODUCTION

This paper is the second part of four papers about "sight-depth".

This paper is about tearooms designed by tea-hosts. In the tearooms, there is a strict rule that determines which direction the tea-host of the tearoom should see and which direction the principal-guest should see with their eyes following the strict tea-ceremony manners in a tearoom.

In the tearoom, there are two spaces which are for the tea-host and the principal-guest. We measure the Sight-Depth of the three-dimensional space for each position and wrote down the space-enclosure as we mentioned in the above part 1[REFERENCE 1]. In this paper, with particular attention to the history and the culture of the age, we evaluate various space compositions by the relation of “transparency” and “opacity” of different architectural spaces starting from the basic tearooms at the dawn in the 15th century to the various tearooms after 16th century. In the enclosure, the visible space that exists as a human environment changes effectively by sunlight inside the space.

In addition to that, we observed the details of the shapes and the positions and ceiling as well as the changes of space-enclosure which are created by the distance between the tea-host and the principal-guest.

After 15th century, Mr. Juko Murata, who first introduced a type of tea ceremony called "Wabicha" and then Mr. Jouo Takeno wrote down the text which showed how to practice the "Wabicha". Since then, the Wabicha was developed era after era. As a result, a kind of tea ceremony for the Wabicha called "Soan no cha" became popular.

Before the tearoom was introduced, the tea-host and the principal-guest had tea in different rooms. But later, the tea-host had tea with the principal-guest in the same room. Having tea in the same room called "Shukyaku Douza" became popular.

After the tearoom was introduced, tea-host Rikyu established "Soanka". His pupils, Mr. Uraku Oda, Mr. Oribe Furuta, Mr. Enshu Kobori and other Samurai tea-hosts introduced the method of showing the relationship of the tea-host and the principal-guest in the tea ceremony indirectly, by utilizing the visual space in the tearoom.

With the historical background of tea ceremonies, after tea-host Rikyu established the tea ceremony, we focus on the various relationships of the positions between the tea-host and the principal-guest. In addition to that, we clarified the meaning of their sitting position in visual space in the tearoom. We also try to have a visual evaluation of the mutual relationship between the tea-host and the principal-guest in the tearoom.

2. THE POSITIONS OF TEA-HOST AND PRINCIPAL-GUEST

A tearoom basically has a partition and also has a fireplace which varies in size. We can find out almost all the composition of the components and the images in a tearoom by taking a look at the location of a partition and the fireplace in the tearoom. The tearoom consists of a tea-host's position and a principal-guest's position. A fireplace plays an important role as the connecting object between the tea-host's position and the principal-guest position. The size of the fireplace is the main element of how sophisticated the tearoom functions and works.

The fireplace is in the tearoom creates a sense of quietness in the tearoom. A basic tearoom has four and one-half Tatami mats in it. "Sumikiri" means that the fireplace is located in the side of the tearoom. "Mukougiri" means that the fireplace is located on the right side of the principal-guest's position. "Daimegiri" means that the fireplace is located very close to the principal-guest's position.

The fireplace is usually located very close to the principal-guest's position on purpose. Tea-hosts pay attention to his own position by considering the location of the fireplace.

3. THE CONCEPT OF SIGHT-DEPTH

In this study we define "sight-depth" as a measure of the distance of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans. Then we use the mathematical definition to describe the perception in the architectural space in computer as we mentioned in the above part 1[REFERENCE 1].

4. THE SPACE-DESCRIPTION OF THREE-DIMENTIONAL SPACE

In this study we define cylinder-drawing, cone-drawing and sine-drawing as a space-description include the distance and angle information of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans as we mentioned in the above part 1[REFERENCE 1].

5. DEFINE OF "TRANSPARENCY" AND "OPACITY"

In this study, we define "transparency" as a component to take in sunlight like plain glass, glass block and so on in the architectural space. Transparent object divides the interior design and the exterior, and divides the space in the building. Especially, the wall is far more important, and often started from the arrangement plan of the wall by the architect in an architectural design in deciding the capacity of the space.

We define "opacity" as a component not to take in sunlight like brick, concrete, wood, stone and so on. The wall, the roof, the floor, and the ceiling and so on divide the space. The factor concerning the development of the lighting is the glass as the opacity material of light as well as the structure. The glass is an extremely old material. It had already been used as a window material in Romanic etc.

6. OBSERVATION OF 32 JAPANESE TEAROOMS

There was no window in the tearoom in the early before Rikyu's establishing the Soan style. Light had been taken from Shoji (=paper sliding door) at the entrance.

Therefore, the degree of the indoor brightness was greatly ruled in the direction of the tearoom. Joho Murano was recommended for the north, and a lot of tea host followed it.

The tearoom changed into the form of enclosing with the wall of the soil, and opening the window when Rikyu Sen made the tearoom Soan style. The degree of light and shade can be freely invented with the size, arrangement, and the material of the window. Rikyu Sen liked space to which brightness was considerably controlled. Only there are small windows on the side of Host and the side of Guest, and a very dark space.

Next, we describe the window of tea hosts after Rikyu Sen.

Joan by Uraku Oda obviously has a lot of windows compared with the tearoom by Rikyu Sen. It is similar for Oribe Furuta. There is no windowless wall. Enshu Kobori opened four windows in the space of only 3/4 mats in the tearoom. Tea hosts of the same Soan style brought up interior space different from Rikyu by distributing a lot of windows like this. However, the role of the window is not only lighting. The opening and shutting of Shoji also has working of ventilation and ventilation. Closing space is formed with the enclosure with the wall. The area of the wall large emphasizes the character of the close.

In the tearoom by Oribe Furuta and Enshu Kobori, the window was used for not only the lighting, ventilation, and ventilation but also sceneries in the room, and, in addition, used for the interior space to relax. Rikyu Sen also used the window to create the space of the tea ceremony greatly and deeply. However, Oribe Furuta etc. were used to express the function of the window more variously. If the tearoom by Rikyu is compared to the monochromatic ink painting, the tearoom by Oribe Furuta and Enshu Kobori can be called interior space at which the color light like the stained glass has been shot.

There are effective devices in the number and how to open the window when the feeling of open and the relaxation of the interior design space are requested. In the tearoom by Sekishu Katagiri, the panoramic window is fully almost opened to the Guest side. Moreover, tall entrance by Shoji for Guest is entrance, and also light has been taken. After modern ages, such a tall entrance is often used. As a result, the atmosphere of the tearoom has lightened.

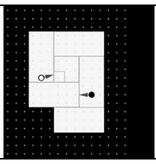
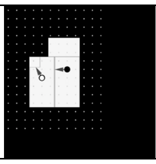
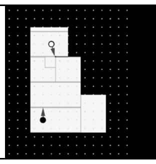
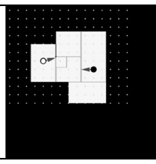
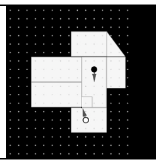
7. CONCLUSIONS

The interior lighting performs a three-dimensional effect with the window. The arrangement of the window will automatically shape the design of the wall. As a result, the composition to which both the function and beauty are mutually corresponding is born.

Rikyu Sen esteemed a natural uniting the function and beauty, and did not open the window so much.

Uraku Oda, Oribe Furuta, and Enshu Kobori, etc. after Rikyu Sen opened a lot of windows. They intended an optical window and the distance of the space arrangement and the effect as the scenery as appreciation were intended.

Table 1: 2. The positions of tea-host and principal-guest at 32 tearooms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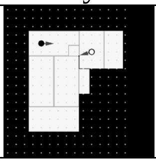	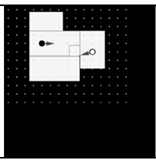	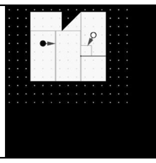	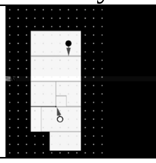	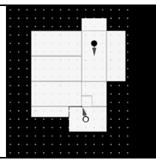
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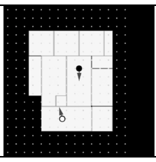	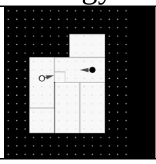	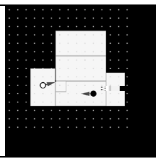	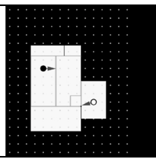	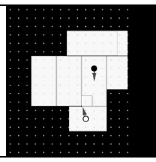
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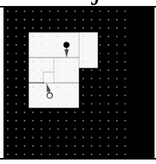	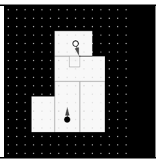	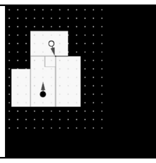	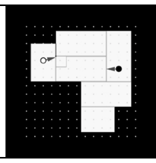	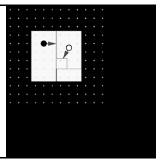
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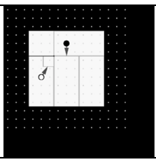	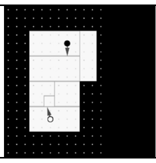	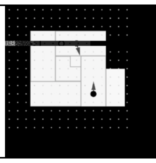	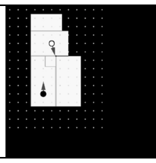	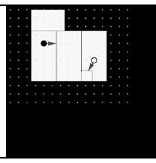
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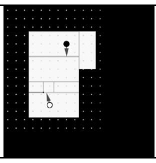	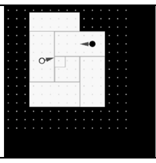	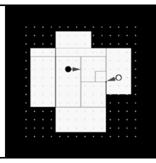	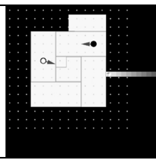	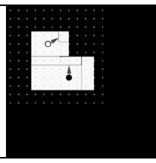
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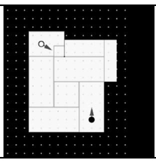	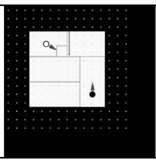	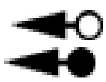 Host position and sight-direction Guest position and sight-direction		
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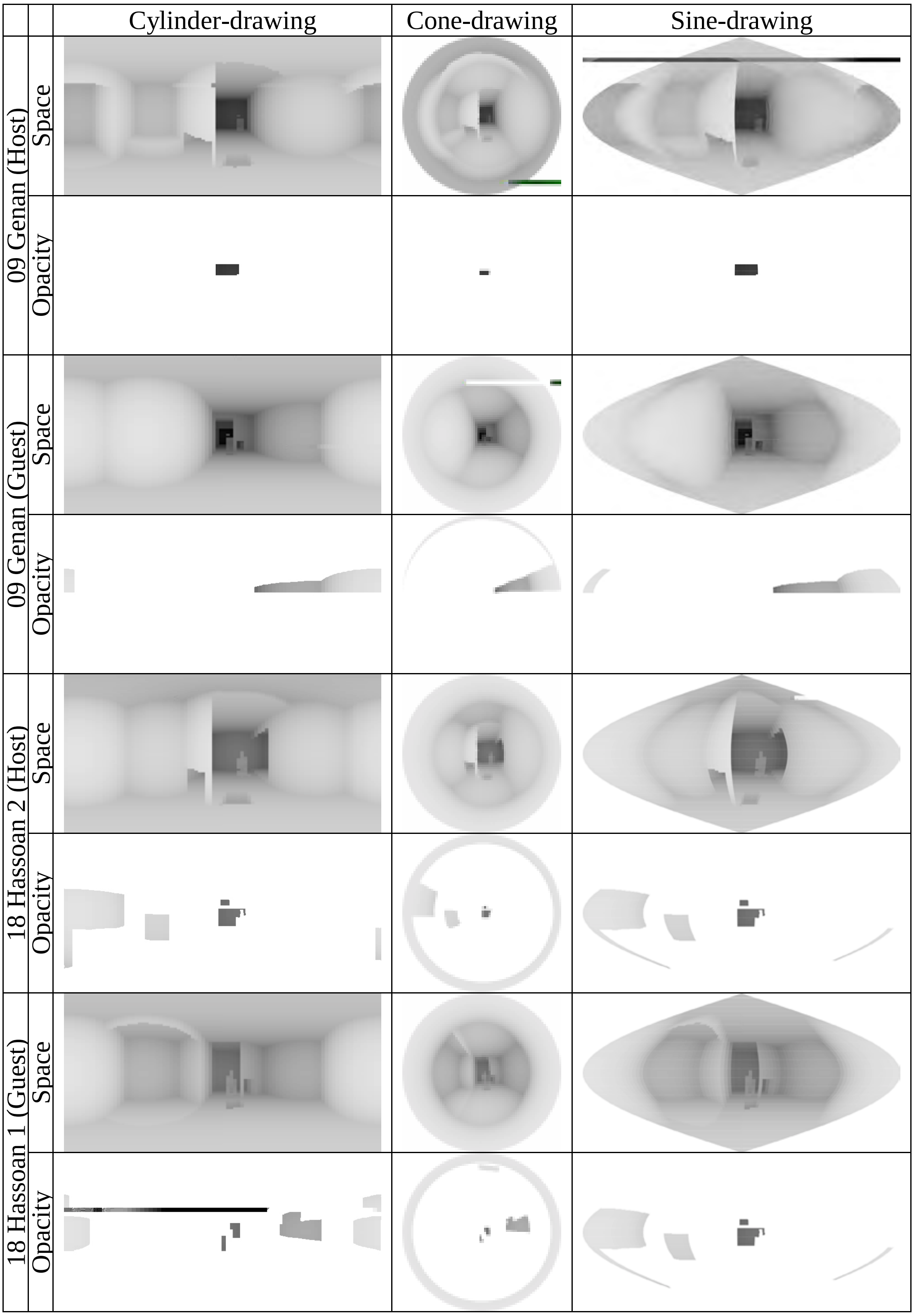

Figure 6: Example of space description of tearooms

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We would like to thank Professor Kyugaku Kato and Professor Sokun Kato who is a tea-ceremony master in "Tea Hisada-school" for kind instructions about tea from the beginning to the end of this research.

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SPATIAL COMPOSITION OF THE TRADITIONAL ARCHITECTURE IN CONSIDERATION OF SEQUENCES

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ABSTRACT

Architecture as inheritance and communication are old no better than the history of the character. It has realized civilization and life-style at each era and has expressed human ideology by geometry like an absolute symbol on pyramid, view of the world like rules of perspective on Brunelleschi and so on.

In this study, we focus on spatial composition as a human enclosure, for example the distance of sight of objects from human, to express the components like wall, column, window, and so on in the architectural space. Then we use the mathematical definition to describe the spatial composition in the architectural space by computer as a new interpretation.

In this paper, with particular attention to the history and the culture of the age, we evaluate the sequences of different architectural spaces starting from the typical large structures like pyramid and dome to structures for religion like churches, mosques and temples. In the enclosure, the visible space that exists as a human enclosure changes with the movement of the standpoint inside the space.

By investigating the transition of the characteristics of the space in the traditional architecture and considering the sequences with walking in the space, we have studied in detail how people have designed the shape of space as an idea and how the style of space composition has changed in each era and the shape of space has spread in traditional architecture.

1. INTRODUCTION

This paper is the third part of four papers about “sight-depth”. In this paper, with particular attention to the history and the culture of the age, we evaluate the sequences of different architectural spaces starting from the typical large structures like pyramid and dome to structures for religion like churches, mosques and temples. In the enclosure, the visible space that exists as a human enclosure changes with the movement of the standpoint inside the space.

Then, this research aims at proposing the method of predicting visual sequence in three-dimensional space by using a personal computer objectively. First, we catch quantitatively the space composition which becomes invisible from time to time in case a worshipper moves in interior design space. Next, we describe a visual sequence by measuring and displaying a distribution of sight-depth. These space descriptions show changes of the visual amount of information of space which are experienced at the time of movement.

By investigating the transition of the characteristics of the space in the traditional architecture and considering the sequences with walking in the space, we have studied in detail how people have designed the shape of space as an idea and how the style of space composition has changed in each era and the shape of space has spread in traditional architecture.

2. THE CONCEPT OF SIGHT-DEPTH

In this study we define "sight-depth" as a measure of the distance of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans. Then we use the mathematical definition to describe the perception in the architectural space in computer as we mentioned in the above part 1[REFERENCE 1].

3. THE SPACE-DESCRIPTION OF THREE-DIMENTIONAL SPACE

In this study we define cylinder-drawing, cone-drawing and sine-drawing as a space-description include the distance and angle information of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans as we mentioned in the above part 1[REFERENCE 1].

4. DEFINITION OF SEQUENCES IN THE HISTORICAL ARCHITECTURE

In this paper, three points on the line of flow from the gate to the central space of the historical architecture are set as the research object in treating the sequence of the interior space of the historical architecture.
First, the first measurement point is a point in front of the gate from external space to internal space. Gate is a opening which passes in case we moves in the interior space and the exterior space which are divided on the boundary

and an element dividing the continuous space, in order to give a turning point in the space.

The second measurement point is a center point on the path between the gate and the altar. Path is a passing way and is the connection space which followed the line.

The third measurement point is the most important and central point of interior space. In the historical architecture, the viewpoints according to the purposes of the architecture such as the religious purpose or the purpose as a racial symbol exist.

As a sequence of the historical architecture, we measure sight-depth from such three measurement points quantitatively and treat them mathematically.

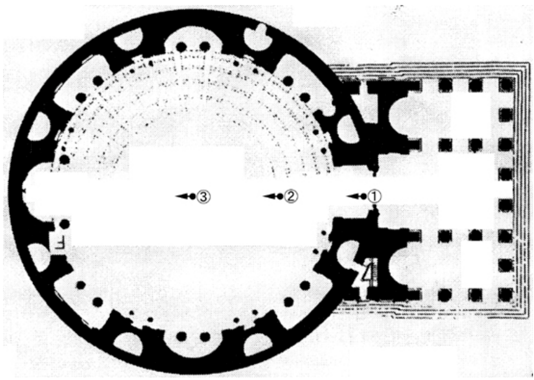

Figure 1: Three points to measure sight-depth of 02 Pantheon

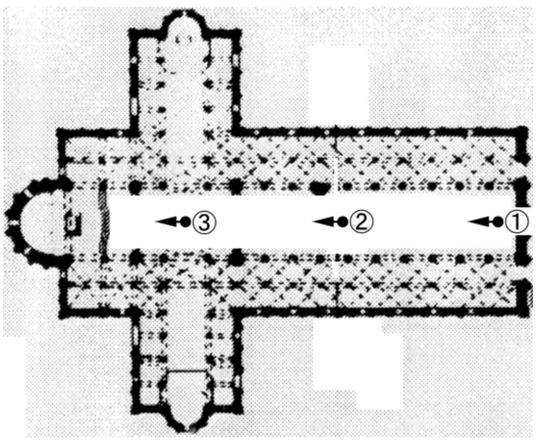

Figure 2: Three points to measure sight-depth of 12 Pisa Cathedral

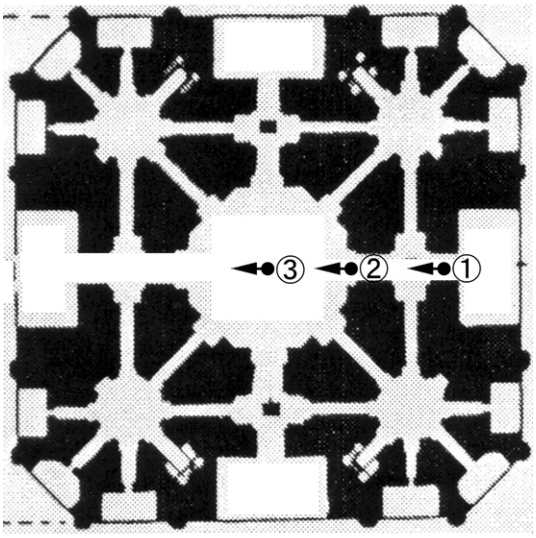

Figure 3: Three points to measure sight-depth of 42 Taj Mahal

5. COMPARATIVE OBSERVATION OF SEQUENCES OF 5 TRADITIONAL ARCHITECTURE

02 Pantheon in Rome is the best-preserved major edifice of ancient Rome and one of the most significant buildings in architectural history. It is an immense domed cylinder fronted by a rectangular colonnaded porch. The entire structure is lit through one opening in the center of the dome.

At the center point, while long sight-depth is constituted widely up, altars extend far back on all sides and are arranged. However, on the gate point and the path point, the range of sight-depth of the dome which existed only ahead at first is prolonged right and left as if aurora spread.

05 Hagia Sophia in Istanbul is most famous Byzantine architecture in Constantinople. It was built from 532 to 537. The half-dome and the small half-dome of both sides of east side and west side, some arches and four huge columns distribute the load of the large dome on the pendentive dome.

At the center point, while sight-depth becomes long abruptly at the upper narrow range, the process in which vault on all sides faces worshippers is often remarkably expressed. The columns of both the sides at the gate point makes sight-depth small as if it emphasized the sequential space which reaches the path point.

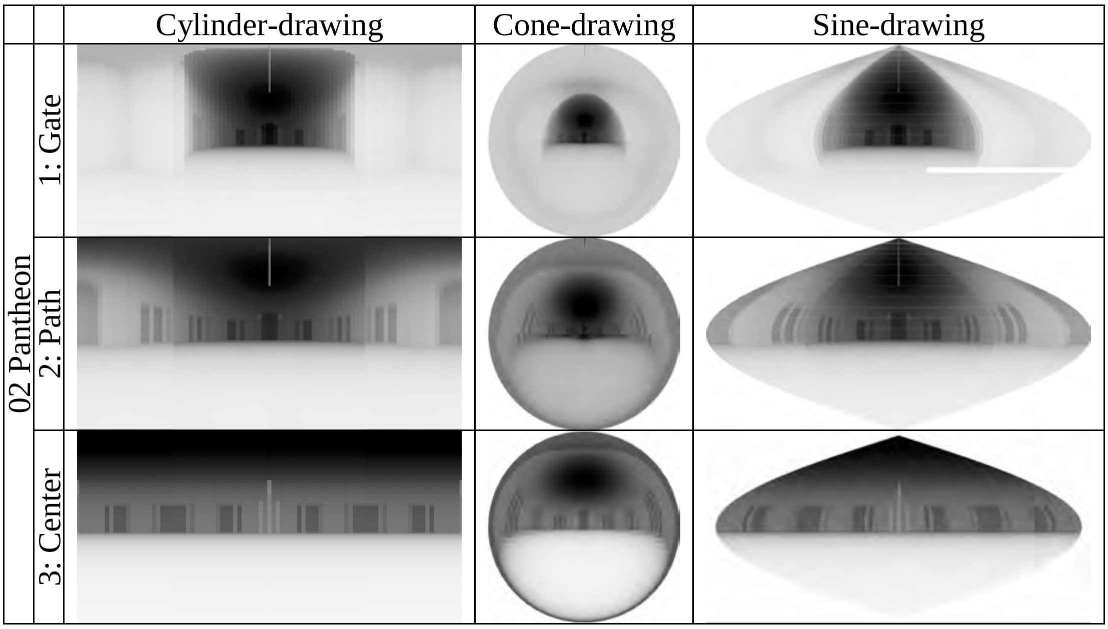

Figure 4: Space description of 02 Pantheon

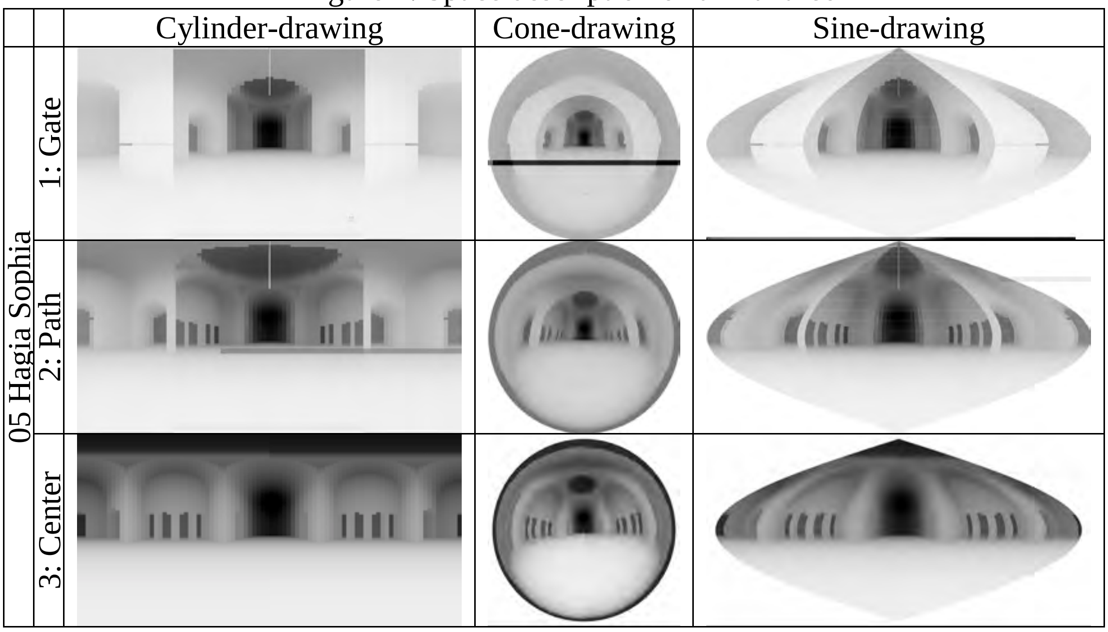

Figure 5: Space description of 05 Hagia Sophia

12 Pisa Cathedral is the typical architecture of the Romanesque architecture and Latin cross plan which consists of five main aisles and three transepts.

Continuation of wall-arches which are typical at the Romanesque style is expressed as deployment of one big arch in space description. At the center point, the space where worshippers have approached becomes deep back.

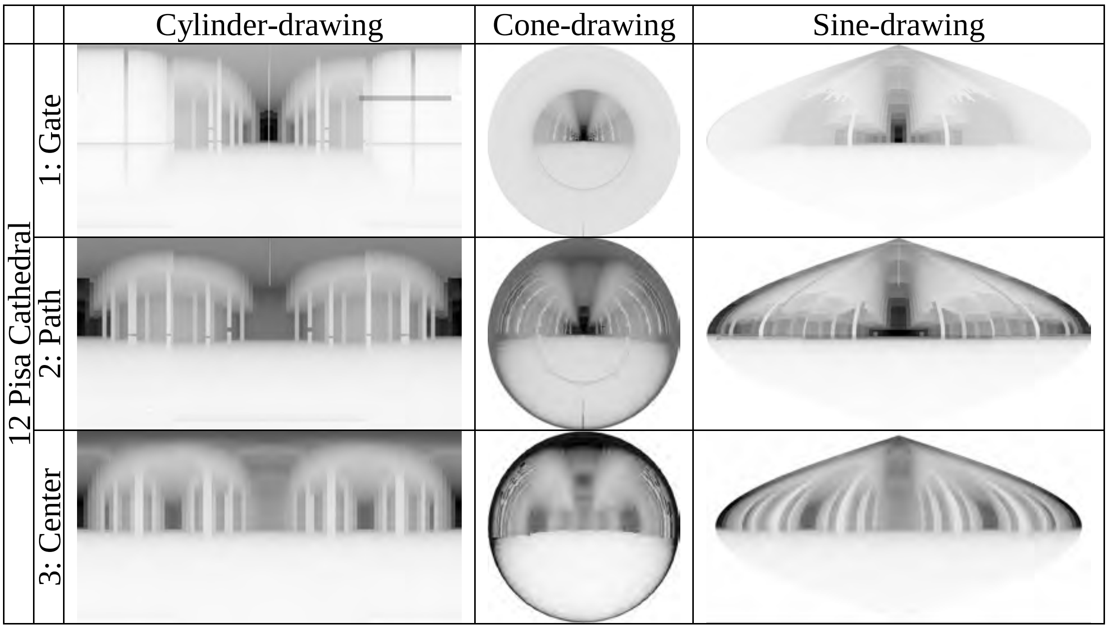

Figure 6: Space description of 12 Pisa Cathedral

16 Worms Cathedral locates in the Rhine River coast and is one of the three cathedrals as well as Speyer Cathedral and Mainz Cathedral and the scale is the smallest among three cathedrals.

Although it is similar to the space description of 12 Pisa Cathedral, it is comparatively simple composition. At 12 Pisa Cathedral, the columns have overlapped intricately. On the contrary, the space between columns is constituted by simple sight-depth at 16 Worms Cathedral. Especially, at the center position, sight-depth does not change on perpendicular direction and horizontal direction. It is similar to the sequence at the interior space of basic rectangular space and consists of very fundamental space.

42 Taj Mahal in India is a red and white sandstone and marble mosque, meeting hall, and mausoleum built by the Mughal emperor Shah Jahan, now 56, for his favorite wife Mumtaz Mahal who died in childbirth some 17 years ago at age 34 after bearing him 14 children.

At the gate point and the center point, symmetrical space is constituted on all sides. At the path point, the objection of sight-depth on either side shifts with wrapping spread of space. Under these circumstances, there is little change of the distance of sight-depth.

		Cylinder-drawing	Cone-drawing	Sine-drawing

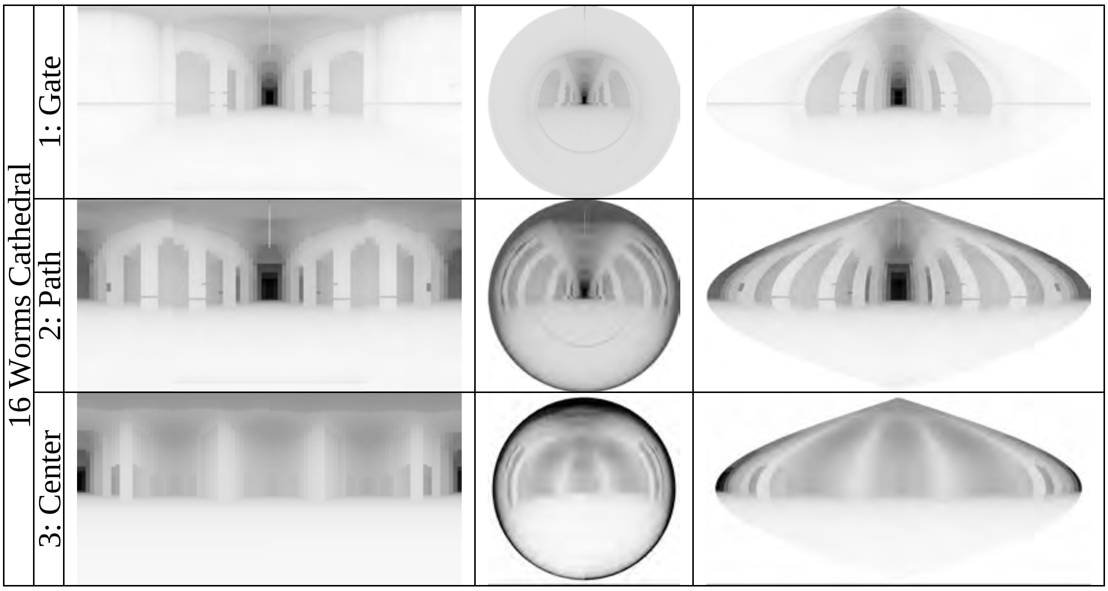

Figure 7: Space description of 16 Worms Cathedral

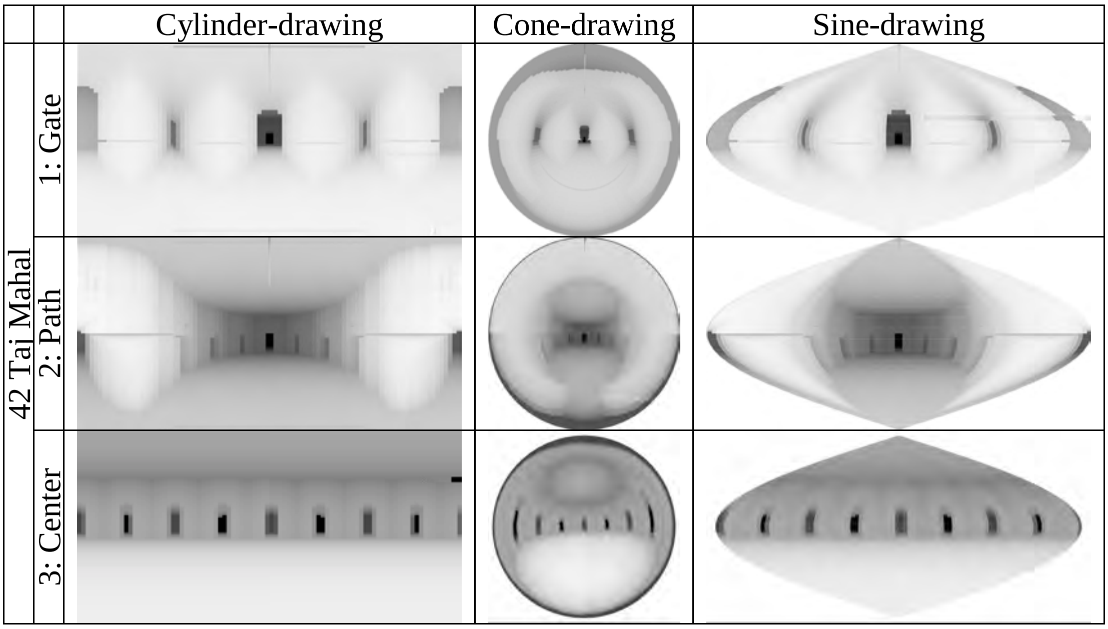

Figure 8: Space description of 42 Taj Mahal

6. CONCLUSIONS

The effect which can be obtained in the structure can be efficiently made with the existence of people in the structure. The proportion in the structure is the size of the proportion between the entire part and each part in the structure.

The structures in early days have relatively simple emphasis in the mathematically proportion of space which occupies the inner space of the structures. The structures constructed later have dynamic styles with rhythmical space elements. The size of the structures is measured in the size of people. Although the structure itself is not very big, the size of the object, such as a step of stairways which can not be measured without considering the size of a person.

In this paper, we described the 52 historically important structures in space-description of sequences by the results we obtained from comparing each of the structures. The structures build for religious purposes are especially huge and we confirmed that such huge structures gives the dignity in scale to the ordinary structures located around them.

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SPATIAL COMPOSITION OF JAPANESE TEAROOM IN CONSIDERATION OF SEQUENCES

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ABSTRACT

In this paper, with particular attention to the history and the culture of the age, we evaluate the sequences of different architectural spaces starting from the basic tearooms at the dawn in the 15th century to the various schools of tearooms from 16th to 19th century. In the enclosure, the visible space that exists as a human environment changes with the movement of the standpoint inside the space.

By investigating the transition of the characteristics of the spatial composition and considering the sequences by rules at each tea ceremony, we study in detail how architects of tearoom have designed the shape of tearoom as an idea and how the style of tea ceremony has changed in each school of tea.

In the tearoom, the change in the position and in the form of opening, partition, ceiling arranged in that space changes the extent of the sight-environment. Once the rule that governs the tea ceremony is confirmed and the enclosed space is kept unchanged and spatial objects are fixed, any change in the host and main guest's position and eye direction changes the sight-environment. We focus the change of sight-environment at each tea ceremony include rules for host to make tea and for main guest to drink tea is the expression of the designer. Movement in the space makes changes of sight-environments in each position. In the space design of the tearoom, strong effect of the design intention of each designer of schools has been confirmed.

1. INTRODUCTION

This paper is the fourth part of four papers about “sight-depth”.

This paper is about tearooms designed by tea-hosts.

In the tearooms, there is a strict rule that determines which direction the tea-host of the tearoom should see and which direction the principal-guest should see with their eyes following the strict tea-ceremony manners at tea-ceremony.

In the tearoom, there are two spaces which are for the tea-host and the principal-guest. We measure the Sight-Depth of the three-dimensional space for each position at tea-ceremony and describe space-enclosure as we mentioned in the above part 1 [REFERENCE 1].

By investigating the transition of the characteristics of the spatial composition and considering the sequences by rules at each tea ceremony, we study in detail how architects of tearoom have designed the shape of tearoom as an idea and how the style of tea ceremony has changed in each school of tea.

With the historical background of tea ceremonies, after tea-host Rikyu established the tea-ceremony, we focus on the various relationships of the positions between the tea-host and the principal-guest at tea-ceremony. In addition to that, we clarified the meaning of their sitting position in visual space at tea-ceremony. We also try to have a visual evaluation of the mutual relationship between the tea-host and the principal-guest at tea-ceremony.

2. THE FLOW OF TEA-CEREMONY

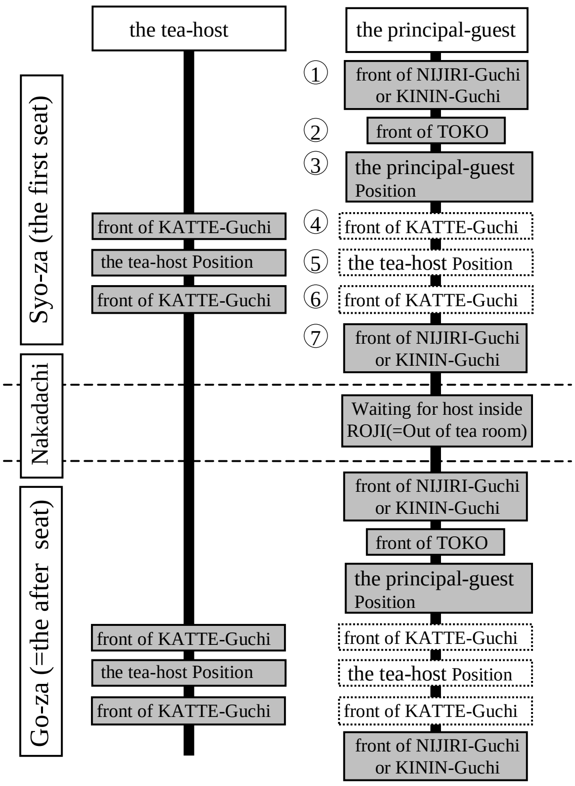

Figure 1: The main flow of the tearoom

The following process holds the tea party. Firstly “Syo-za” means that the principal-guest gets tea from the tea host is held (=the first seat). Secondarily “Nakadachi” means that tea host is waited for as the principal-guest has a seat in the outdoors. And at the end “Go-za” means that the principal-guest gets the entering meal again in the tearoom is held (=the after seat). The two people’s movement in the tearoom is almost considered to be the same in the first seat and the after seat, and the flow of the tea party in the first seat is targeted in the analysis in this research. The main flow of the tearoom is shown as follows. (Figure 1)

Previously first the principal-guest is entering from small entrance for guest (=NIJIRI-Guchi) or normal entrance for guest(=KININ-Guchi)—1 , the principal-guest who entered the tearoom moves in front of the “TOKO” while sitting straight and sees kakemono—2. Afterwards, principal-guest waits for the tea-host sitting on a fixed position—3.After a while the principal-guest enter the tearoom from kitchen door for host (=KATTE-Guchi)— 4 ,who starts making powdered green tea— 5 . When a series of entertainment ends, the tea host previously leaves the tea room — 6 . And, a positive guest makes the tearoom a back at the end— 7 .

To the above-mentioned way, Sight-depth of ten aspects of three tea-host’s aspects and seven principal-guest’s aspects in total was measured per each of the one tearoom.

3. DEFINITION OF THE TEA-HOST POSITION AND THE PRINCIPAL-GUEST POSITION

The basis of a plane composition of the tearoom is the room arrangement and how to make a sunken fireplace in the floor, as a result, the composition and the peculiar characteristic of the tearoom can be almost guessed. The inside consists of the tea-host’s tatami mat space (=temaeza) and the princilpal guest’s tatami mat space (=kyakuza). It is a fireplace to do the role for temaeza to tie to kyakuza. How to make a sunken fireplace in the floor is a definite element that controls the role and the peculiar characteristic of the tearoom. Various hows to make it has come out to “Sumikiri" which make it to turn back to guest and “Mukougiri" which moved it to the guest vicinity of eyes and “Daimegiri" by which was make to kyakuza side. The fireplace was made near guest's eyes, and the concern was poured like this how to show entertainment. United the following rules were provided about each position of the tea-host and the principal-guest and the direction according to information obtained from the drawings of tearooms such as the fireplace and the small entrance in this research though some differences were seen at the position of the tea-host and the principal-guest in the tearoom because some sects and manners existed (Figure 2).

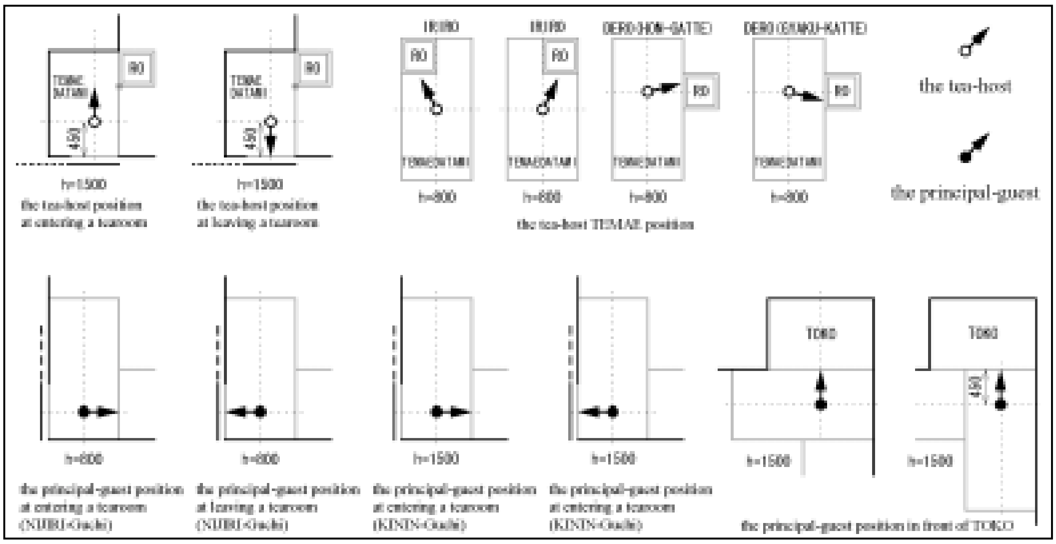

Figure 2: The rules at the position of the tea-host and the principal-guest

[In front of doorway the principal-guest position – view 1 and 7]

It turns in the direction shown from on a center line in the doorway and the doorway in figure at the position of 1.5 shakus (450mm) in front of the mall entrance or the normal entrance.

[In front of TOKO the Principal-guest position- view 2]

It turns from the intersection in a centerline of TOKO a centerline in the mat to TOKO when mats are longer than the width of TOKO.
When the width of TOKO is longer than that of the mat, it turns from the position of 1.5 shakus (450mm) to TOKO the edge of a centerline in the mat.

[The principal-guest's fixed position – view 3]

The principal-guest arranges it on a centerline of the guest’s mat.

The position of a small entrance and a normal entrance is considered respectively.

It sits on the position where entertainment and TOKO can be basically seen well in the guest’s mat, and it orthogonalizes to the mat and it turns to the tea-host.

[In front of tea-host’s kitchen door – view 4 and 6]

It turns from kitchen door on a center line of kitchen door in the direction shown from the position of 1.5 shakus (450mm) in figure.

[The tea-host fixed position – view 5]

It arranges it on a centerline of the tea-host’s mat.

The position of kitchen door is considered respectively.

The tea-host turns from the fireplace to the corner in one with a far fireplace for Iriro (it means that fireplace is in the tea-host’s mat) because he parted from three suns (90mm).

The tea-host sits on the intersection in on a centerline of the tea-host’s mat and a centerline in the fireplace for Dero (it means that fireplace is out of the tea-host's mat). The direction is changed by Hon-katte (= the formal direction) and Gyaku-katte (=the inverse direction), and it turns to the corner in one far from the fireplace.

4. THE CONCEPT OF SIGHT-DEPTH

In this study we define "sight-depth" as a measure of the distance of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans. Then we use the mathematical definition to describe the perception in the architectural space in computer as we mentioned in the above part 1[REFERENCE 1].

5. THE SPACE-DESCRIPTION OF THREE-DIMENTIONAL SPACE

In this study we define cylinder-drawing, cone-drawing and sine-drawing as a space-description include the distance and angle information of sight of objects from human, to express the components like wall, column, window, and so on in the architectural plans as we mentioned in the above part 1[REFERENCE 1].

6. COMPARATIVE OBSERVATION OF SEQUENCES OF 32 JAPANESE TEAROOMS

6.1. 02 TAIAN

Compared with other tearooms as the entire flow, the space-enclosure of the principal-guest space is strong, and the change of space-enclosure is loose. And the ratio, which the tea-host and the principal-guest occupy to the entire space, is large. Therefore, the space-enclosure is a strong, narrow space in both of both. But the space from each other to each other begins to have extended when two people (the tea-host and the principal-guest) take both to a fixed position by the effect of the going up ceiling (Figure 3).

6.2. 08 JOAN

As for the positive guest space, there are three secluded which extends soon before the diagonal the left after it enters the tearoom, and TOKO, the tea-host's space, and inclining ceiling are distributed from the left. It is understood well that secluded of the tea-host's space makes TOKO stand out in front of TOKO. It is a space composition to extend right and left like the radiation when taking it to the principal-guest fixed position, for entertainment to be done when the tea-host shows up sideward of secluded of the front, and to make inclining ceiling called "Kesyoyaneura" feel outdoor in the right hand. When entering the tearoom, it was lower right secluded in the tea-host's space. The space done in a narrow tearoom in a relaxed manner by the effect of "Urokoita" (=triangular board) and Katougata (=arch type) which turned over and was pulled out is created to the wood siding wall when moving to the tea-host's mat (Figure 4).

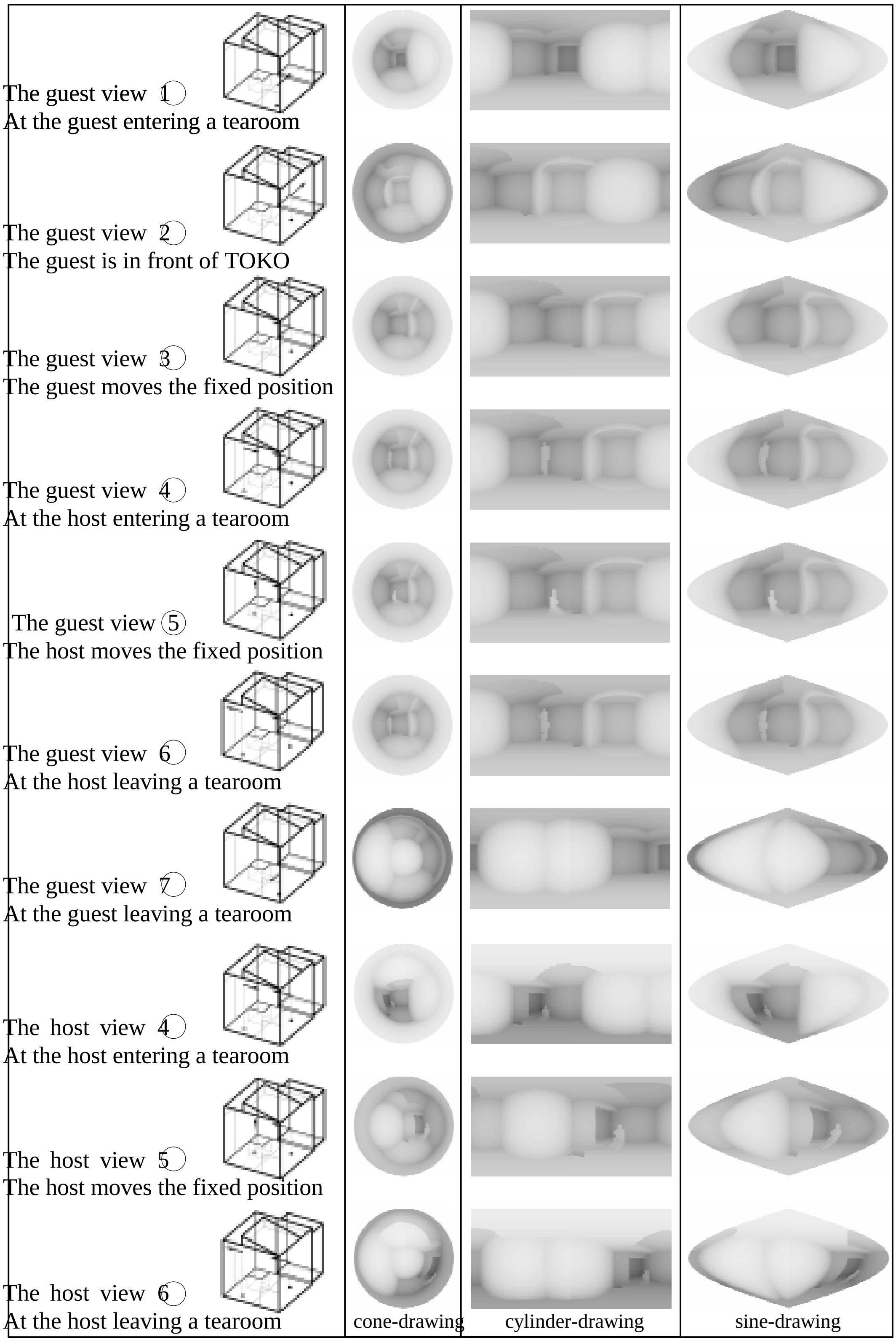

Figure 3: The space-description of 02 TAIAN

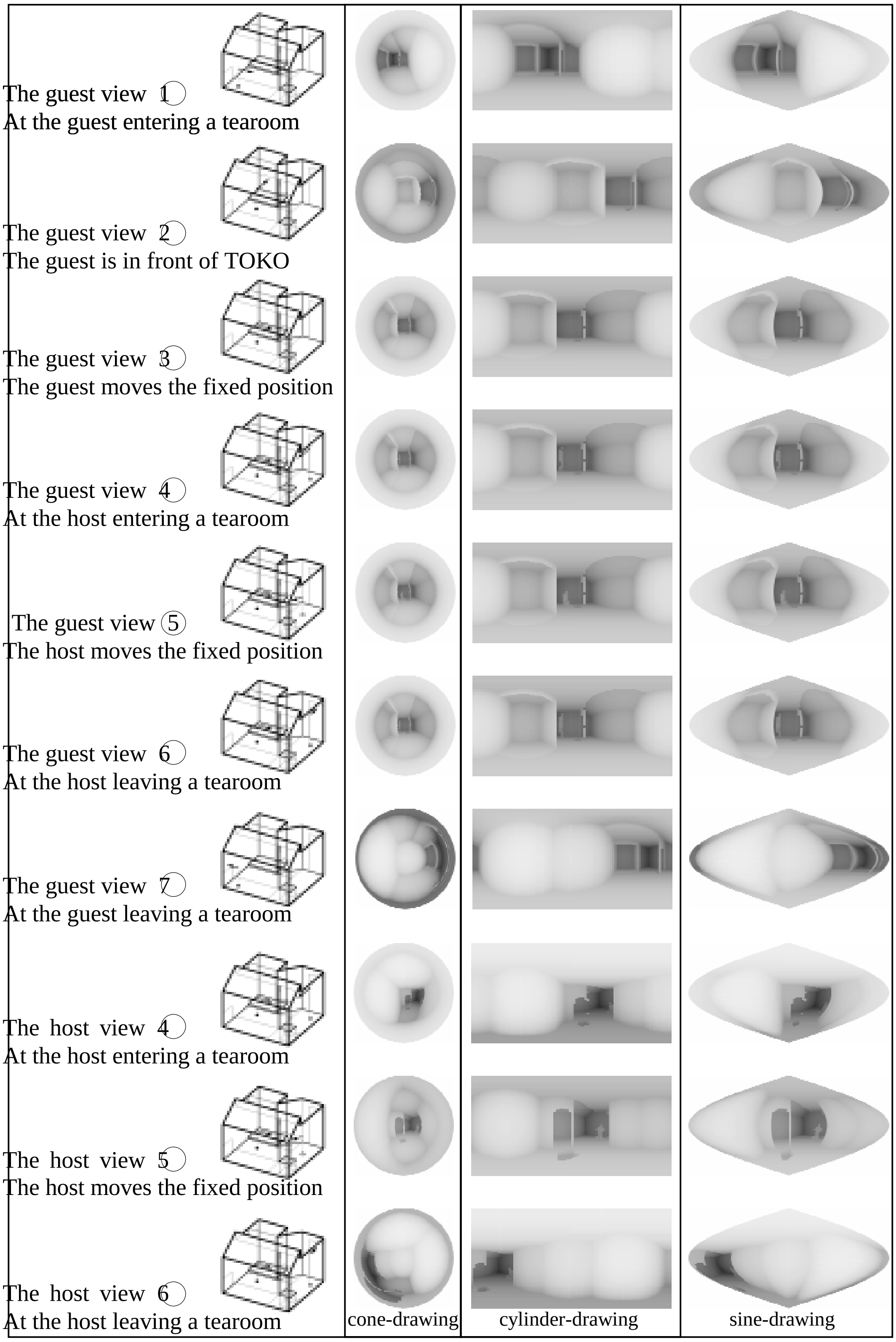

Figure 4: The space-description of 08 JOAN

6.3. 09 GENAN

In the principal-guest space, the change in the space-enclosure in front of the the small entrance and in front of TOKO is extremely violent, it changes completely from the space which expands in the interior, and it becomes the space of the ultra narrowness enclosed by TOKO, Chinkuguri (=partition while it is between the tea-host and the principal-guest), and Ochi-ceiling (=lower ceiling than surroundings). Entering the tearoom is a feature space from which the principal-guest and TOKO are distributed right and left, and a space to which the ceiling is pressed low. However, vertical strongly distributes TOKO behind when taking it to the tea-host fixed position, and it does in a relaxed manner.

6.4. 14 TOSHIN

Secluded is enclosed from the front true back in a little lower right by this level in the principal-guest space in front of TOKO after he enters the tearoom, and it distributes in the right hand at the principal-guest's fixed position and secluded is distributed to the tea-host space having right side front. It is a space composition that the extension of the space consists as consideration still turns to the object also of here. As the principal-guest space the interior taking change in the space is not seen though it moves from secluded of lower right to the front. It is a typical example of the tearoom in both principal-guest spaces and the tea-host spaces.

6.5. 18 HASSOAN

Secluded of the principal-guest space in the direction of the right hand stands out a little though TOKO is received in the front in the principal-guest space after he enters the tearoom. The space composition does not change so much though the principal-guest advances in front of TOKO, there is an extension right and left at the principal-guest's fixed position in some degree, and it is in front the strongest secluded. As the husband space, it is the most typical example of creating the space where extremely vertical when the one that there is secluded in lower right moves to the tea-host's space is strong.

6.6. 25 YODOMI

TOKO is distributed to the front when entering the tearoom, and the tea-host's mat exists as secluded in the right hand in the principal-guest space. Secluded of the tea-host's mat is the strongest when advancing in front of TOKO. Secluded concentrates only on one point for secluded as which the whole is almost the same at the principal-guest fixed position. It is understood that secluded of that just becomes the principal-guest and the tea-host's points of contact when the tea-host enters and he moves to his mat, and each other has concentrated on a direction each other. This is an effect of the partition between named "Douan-kakoi" which Sennodouan produced. It is understood to use the space-enclosure well.

6.7. 32 SEIKO

It is an unusual tearoom where TOKO and the tea-host's mat queue up sideways when seeing from a positive guest, and TOKO is not seen from the tea-host. When entering a room, light makes entering entertainment easy to show the tea-host's mat by sideward of TOKO having become empty a little when seeing from the tea-host though this tearoom does not have a wonderful difference with another.

7. CONCLUSIONS

The act ' Tea was drunk ' that was a part of the feast began originally to become independent only by it, and it came to be concerned. In the tea party done from such a meaning by a minimum space of tearoom, it is thought that the originality device of the master of the tea ceremony has been poured how into the guest can be entertained. As a result by seeing the space-enclosure in the tea-host and principal-guest's each aspect in detail, a remarkable respectively feature, which differed quite, appeared in the space-enclosure of both though it was in the same space.

In the process which moves to the tea-host's mat after the tea-host enters the tearoom, most cases change from secluded under lower right or the left to strong secluded in a small range in the front . The tea-host is to keep from the principal-guest, and the intention of entertaining the guest in the stage named Temaeza appears. A variety of space transitions of each tearoom were clarified about the space-enclosure of the principal-guest. In the tearoom, the minimum space, that the space-enclosure rapidly changes only into whether stand or even sit, one feature there with was found when seeing about the principal-guest who moves in the tearoom according to one manners. The tea-host space existed in the direction when it searched from the principal guest's direction of the glance for the direction with secluded most. This is not a problem that the space is narrow or wide. It can be understood that it is how to show the space from the aspect of how to enclose the space.

Thus it can be said that the tearoom space in the tea party, which progresses according to one rule, will produce the maximum entertainment in a minimum space by the originality device, which the master of the tea ceremony fully becomes.

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AN ANALYSIS OF THE SINGLE DOMED SELJUK MESJIDS IN ANATOLIA

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ABSTRACT

13th century single domed Anatolian Seljuk's Mesjids are outstanding monuments with their plan design, semi opened and enclosed entrance sections, variety of dome transitions and tile mosaic mihrabs. The aim of this paper is to study 13th century single domed Anatolian Seljuk's mesjids that have background of 800 years from the planning, construction systems, material and façades characteristics points of view by comparing them to Anatolian and non-Anatolian structures to find out its contributions to architecture and to figure out its place in Anatolian Seljuk Architecture.

A catalogue for the single domed mesjids built in the 13th century is prepared and detailed information of every building is given : Study in regards to the architectural properties or qualifications, the characteristics and value of the material used, as well as their decorative techniques. Determination of conditions of the structures, a swell as determination of reasons why the structures have deteriorated. Upon evaluation of the catalogue data plan designs, constructions systems, elements and façade forms of the monument are studied and some conclusions are drawn.

The effects of Anatolian Seljuk's mesjids on the plan design of the main space and latecomers' area of the Beyliks and Ottoman mosques and effects on the plan organizations of the dome covered Ottoman architecture is studied and an evaluation of this study is made.

1. INTRODUCTION

The Anatolian Seljuk's opened a new chapter in architecture in the 13th century with their building types, hewn stone monumental architecture and embellishments to enrich space effect. In this century a large portion of the Seljuk mosques constructed in Anatolia are structures with multiple columns and domes in front of the mihrab, like those traditional in Islamic countries. Even if each of these domes in front of the mihrab are considered a small experiment in volume, the structure is far from saving the interior from columns and achieving a compact, broad space. In the 13th century one finds a second structure designed for worship and shows the thoughts that led to the initial steps taken towards a compact area and differs from the previous structural type. In particular these buildings that appear in the capital Konya and its vicinity are mesjids, built in districts to meet the worship needs of small communities. These monuments are enhanced by having their square main space covered with domes, the entrance sections semi opened and enclosed and a variety of dome transitions, plain façade, rich interior decoration, mosaic tile mihrabs and the development of the domed space in Ottoman architecture. They have a separate importance when compared with the monumental mosques and madrasas constructed in the 13th century.

2. PLAN CHARACTERISTICS

It has been proven that 53 single-domed mesjids belong to the 13th century (Table1). And over a period of 800 years these monuments underwent rather significant changes. Thirteen of them(25%) were destroyed. Seven(13%) lost their original form as a result of repair and restoration. As for the remaining 33 structures(62%), they in part preserve the characteristics of 13th century architecture. Mesjids are planned to be square or nearly square with a variance between 4.27m and 8.50 m and have a main space covered by a dome. They are planned rectangularly only if covered with three structural domes. In the Akşehir Kızılca Mesjid the rectangularly planned area is covered with an oval dome and in the Konya Abdülmümin and Beyhekim Mesjids the area is turned into a square through transitional elements in the rectangular substructure. These can be considered the first efforts in the 13th century directed at a broadening of the space. In the mesjids three types of plans were developed : single unit, two units and three units (Figure 1).

2.1. Single Unit Mesjids: Thirteen of the 34 structures(38%) examined have single units. The single unit mesjids became more frequent in the first half of the 13th century but are only met in three places during the second half of that century.

2.2. Double Unit Mesjids : Of the 34 examined, 20(59%) have two units. In these mesjids it is seen that the second unit was added to the front of the main unit (Figure 1). In the 13th century single domed mesjids, the dimensions of the domes are a structure for worship restricted in area because it was not as yet possible to cover a broad space. As a result a second unit was added either to the entrance section in front of the domed main space or as a mausoleum.

Mesjids With Entrance Sections : In the mesjids seventeen of the 20 structures have been designed with entrance sections. The entrance sections that appeared in the 13th century as innovations are either enclosed or semi opened (Figure 1). It is unclear how the particular forms, as well as the entrance sections, of the Konya Aksinne, Ferhuniye and Erdemşah Mesjids and the Akşehir Altunkalem, Güdük

Minare and Hacı Hamza Mesjids occurred. The trail begins from the Konya Hacı Ferruh Mesjid of 1215 that preserves its original form and whose construction date is known for sure. It is thought that the plan was for the entrance units to be enclosed and later, when the front façade was opened to the outside, semi open entrance sections were created.

Entrance sections in semi open mesjids; the two side façades are bounded either by being enclosed by the walls of the main space that have been extended straight forward (Cat. No:7, 37) or by one of the side façades and the minaret (Cat. No:31, 38, 45, 47, 48). In some examples the front façades of the entrance section enclosed on two sides are open to the outside through arches that rest on columns (Figure 2). In others they are arches resting on pillars or are in a basic lean-to form.

The entrance section, thought to be essential because of the restricted main space, are referred to as “vestibule” or the “latecomers’ area” in some sources [1]. But referring to these sections as the latecomers’ area is not correct for all of them. The directional situation of a portion of the entrance space is neither deep nor suitable for performing the prayer service (Cat.No:13, 31, 45, 46, 48). These sections are places where the worshippers who on entering and leaving the mesjid can find protection against the sun and rain and while waiting for prayer time to begin can engage in conversation. They are a passage that keeps one from entering the mesjid immediately and serve as a place of preparation. Some mesjid entrances are placed in the direction of the mihrab in the north façade of the mesjid so that in terms of depth and shape, the direction is suitable for praying (Cat. No: 7, 15, 37, 38, 47, 49). That the doors opening on the main space are placed on an axis with the mihrab and in some meet the niche itself have given rise to the thought that prayer was held in these units and they were designed as the latecomers’ area. These buildings with entrance sections designed as enclosed or semiopened are the first examples and forerunners of the latecomers’ area.

Mesjids With Mausolea: It is observable that in some buildings second units planned for mausolea have been added to the front of mesjids with single domes. In a third of the 34 buildings examined there is a mausoleum portion adjacent to the main space (Cat.No:30, 50, 52; Figure1). In the Karaman Saadettin Ali Bey and Konya Cemel Ali Dede Mesjids the mausoleum unit opens to the outside in the form of a aiwan. But in the Harput Arap Baba Mesjid, the mausoleum unit is closed to the outside and the entrance to the mausoleum is from the main space. It is striking that the mausoleum unit is located in mesjids despite its being contrary to Islamic traditions. As is known, the building of cemeteries did not develop among Islamic peoples until the ninth century. However pre-Islamic beliefs weren’t suddenly wiped out with the coming of Islam and they continued to have an influence in a new way within this system of belief and thought. In the mausolea of Turkistan and Iran the square plan and the domed type were the most preferred and numerous examples are to be seen. During the time that developments were occurring in Anatolia, the form changed together while preserving the square plan outline that lost its cemetery function and in the enclosed or semi opened entrance sections added to the building and a new structural type appeared.

2.3.Three Unit Mesjids: There is only one structure in the three unit plan outline (Figure1). In the Konya Bulgur Dede Mesjid the semi opened unit in front of the main one and a second unit have been added to the west. How this unit functioned is unknown. The building has a different plan outline because of the mihrab in the entrance area is of monumental measurements and rises from the main unit's floor.

3. BUILDING SYSTEM AND BUILDING ELEMENTS

Materials: One can see that in mesjids the mainly preferred building material is brick. Hewn stone and rough stone are used with brick. However in two buildings, all of the walls are of hewn stone (Cat.No:13, 30). Of these buildings the wall built with very advanced techniques in the Konya Hacı Ferruh Mesjid is an example that shows the high quality of Seljuk stonework. In wall construction composite materials have an important place. In particular in the wall construction of the mesjids in Aksehir composite materials are used to a great extent.

Walls: In mesjids the dome is seated on a square understructure bounded by walls (Figure3). In order for the walls to carry the load of the dome it is essential that the thickness of the walls be no less than a certain limit. In the buildings the wall thickness ranges between 71cm. and 124cm. It was thought that only walls of this dimension could meet the pressure of the weight of the dome as it was dispersed towards the sides. The window apertures were chosen in dimensions that wouldn't damage the walls' ability to carry the weight.

Domes: Two types of roofs are found; the vault in the entrance areas and mausoleum parts of mesjids and domes in the main spaces. The entrance sections that had a special form cannot be evaluated in regard to the vault roofing because they were very few in number. In mesjids, domes cover the square-planned main spaces. There are two mesjids whose domes have been destroyed and which are covered with a plain roof these days: Akşehir Kileci Mesjid, Konya Hacı Ferruh Mesjid (Cat. No:7, 13). The dome has the characteristic of being the main motif and it is not of overwhelming size. One sees that dome dimensions were circumscribed by the static boundaries that construction techniques in the 13th century dictated and the fact that open spaces of more than 8.10m could not be bridged. In six buildings one sees that the height of the dome and the radius are equal and as a result the dome cross sections are half the room or nearly so. The dome is (except for two buildings) seated directly on the walls of the building and passes from the walls to the dome without a drum. One can observe a narrow drum between the lower structure and the dome in the Halka Begüş and İç Karaarslan Mesjids. The use of double domes is not met in mesjids. The domes with one exception are made of brick. A stone dome is only found in one 13th century structure, the Konya Hacı Ferruh Mesjid. The stone dome of the Hacı Ferruh Mesjid has been destroyed. The material used in the dome of the Karaman

Saadettin Ali Bey Mesjid can't be ascertained because it has been plastered over. It is known though that the dome was of stone [2]. Projecting parts of the brick on some of the domes are observable. That there were brick projections on the domes in almost all the mesjids is observable in the photographs of buildings in past years. These projections can't be seen because most of the domes have been covered with lead. One portion was destroyed during repairs. It is thought that, although the reason for the projecting bricks has not been ascertained, it may have been to facilitate going wherever necessary on the dome. Holes are observable where the dome starts on some mesjids, in the dome's transitional elements and inside the wall fabric (Cat.No:8, 20, 35, 36, 38, 47). The forms and depths of these holes in the fabric cannot be assessed. However the guess is that the cubes placed in the fabric are there to solve the acoustic problems of the structure. In Ottoman mosques too advantage was knowingly taken of cubes in order to provide for a homogeneous distribution of sound. The cubes, located in a way in which their edges would be open towards the inside, would act as a resonator in the empty space and solve the acoustics problem.

Dome Transitional Elements: The transitional elements in mesjid domes are in three different kinds; pendentives, trompe and triangular bands. One observes that the transitional element most preferred as it was in the Great Seljuk period was the trompe and it was rather plain. The pendentive was only used in two structures. These two examples show that in Seljuk mesjids the pendentive was not a transitional element on the highest degree. In ten structures it is observable that the triangular band and various forms of it were tried.

Mihrabs: Stone, tile and plaster of Paris were used in mihrabs that were the most important element in the main space in mesjids. In three buildings hewn stone is found in a mihrab. Of these, the Hacı Ferruh Mesjid mihrab is an important example which reflects early Seljuk stone work in Konya. As for the tile mosaic mihrabs, they are elements which catch one's eye at first sight in the kible wall and in the entire main unit with their colorful and shining surfaces. The tile mosaic technique, met in Anatolian mihrabs in the 13th century for the first time, offered the opportunity to enclose curving surfaces. This is an innovation which the Seljuk's brought.

Minarets: Minarets generally rise on hewn-stone square bases. The positioning of the minaret at the entrance façade in the 13th century is still not definitively known any more than there is a specific rule about its being on the left or the right of the entrance. However there is an attention-getting characteristic in that the minaret does not have a place at closed entrance units in contradiction to its being found in mesjids whose entrances are open. Double balconies are encountered in four minarets (Cat.No:12, 31, 47, 48). The one example still standing of those with two balconies is that of the Akşehir Taş Medrese (Figure 4). While single balcony minarets were made in the great mosques and medreses which the Seljuk's constructed in the 13th century, it is surprising to see that there are double balconies in such small mesjids and then only in Konya and its vicinity.

Façades: There are no strict rules about the façades of mesjids (Figure 5). Aside from the front façade no special design effort is observed and the design of the interior space is reflected in the façade. Embellishment is used to stress structural elements in a balanced way without excess but related to the architecture. The windows and doors are of such dimensions that they don’t spoil the weight-bearing character of the walls. Just as they can be very plain and simple, sometimes one meets places decorated with tiles and enamelled bricks. The windows had no particularly specific location. They were situated symmetrically on either side of the mihrab or the door just as they might be right in the center of the façade or shifted to the side. The doors and windows were of such different flat arches, arches and discharging arches. It’s not possible generalize about specific types being used more often at specific periods for the doors and windows. However in the second half of the century the number of windows increased and the surfaces broadened. With the introduction of a construction program for entrance sections, the monumental portal lost its importance in Seljuk architecture and doors were simplified as the form changed within the façade organization. The continuation of the traditional portal is only seen in two buildings which date from the beginning and the end of the 13^{th} century. The entrance portal of the Konya Hacı Ferruh Mesjid can be considered an exception putting it together with the monumental main door seen in other buildings among the 13^{th} century Anatolian Seljuk’s; it is not met in the mesjids (Figure 6). In the entrance door area of the Konya Sahip Ata Mesjid built at the end of the 13^{th} century and whose special form today has been completely ruined, the tendency to achieve portal on a small scale through various techniques such as the use of enamelled bricks and geometric design is observable. Another innovation is in the minarets at the entrance façades. The minaret in the 13^{th} century in the semi opened entrance sections is a structural element continually used. The form of the façade is completed with a minaret adjacent to the entrance section either on the right or the left of the façade.

4. CONCLUSIONS

According to extant examples single domed mesjids accepted in the 13^{th} century in Konya, the capital of the Seljuk’s, and its vicinity have characteristics which permit considering them experimental.

For the first time the area used by latecomers in Anatolian mosque architecture appeared systematically in Seljuk mesjids along with enclosed or semi opened entrance sections and while a functional latecomers’ area as an architectural motif was prepared at the same time, the basis was laid for the design of a different façade. In the 14^{th} century quite a few mosques were completed with façades shaped by minarets placed in the northeast or northwest corner and the leader in these types of mesjids was again the Seljuk mesjids (Figure 7). In the 14^{th} and 15^{th} centuries various forms were tried out for the latecomers’ area. Researchers have accepted that the idea for a place for latecomers arose in the Beylik period in

Anatolia, the first examples of such are shown as the mosques in 14th and 15th century İznik, Bursa, Milas and Kütahya [3]. In the İznik Hacı Özbek(1333) and the Hacı Hamza(1345) Mosques at the beginning of the 14th century, the entrance units not on the mihrab side show that the idea of a functional latecomers' area was still in the creation stage and the Seljuk tradition had not ended.

While small dimensioned but sound spatial experiments were carried out in Anatolian Seljuk mesjids, the lead was taken by the single domed mosque type thus providing for unity of space under a single dome (Figure 8). The outline plan of the mesjids that were the first products of the idea to increase space and gather it under a single dome became the leader in new spatial developments through the strengthening of technical possibilities in the 14th century. Various examples of the single domed structure were tried and almost at the same time that mosques resembling each other in character were constructed, efforts were being made to broaden the space and to solve the problem of enlarging the diameter of the dome. At the beginning of the 14th century the dome dimensions were 8.00m. in the Bursa Alaeddin(1326) and İznik Hacı Ozbek(1333) Mosques. In the middle of the century it had risen to 12.00m in the Gebze Orhan Mosque and at the beginning of the 15th century it had become 14.50m in the Balat İlyas Bey Mosque(1404). During the 14th century in the Mudurnu Yıldırım Bayezit Mosque(1388), it is observable that the dimensions of the dome had grown even more and were now 19.65m. Ayverdi indicates that this experiment was a structural experiment for the dome but was not advantageous for architecture [4]. All these efforts prepared the way for the classical mosque type of the Ottoman period with its single dome and latecomers' area in three sections. The dimensions of the dome covering the main space with its square plan in Seljuk mesjids was 8.10m and it grew in the Beylik period to 19.65m, expanding as time progressed. In the 16th century it had reached monumental dimensions with measures reaching 31.50m.

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THE SINGLE DOMED SELJUK MESJIDS

CAT NO.	MESJIDS	DESTROYED	LOST ORIJINAL FORM	PRESERVE ORIJINAL FORM
1	KONYA HOCA YUSUF MESCİDİ	•		
2	KONYA SİNCARİ MESCİDİ	•		
3	KONYA ŞERABİ HACI İSA MESCİDİ	•		
4	KONYA EB-ÜL FAZL MESCİDİ	•		
5	KONYA YUSUF BİN SÜLEYMAN MESCİDİ	•		
6	KONYA AKSİNNE MESCİDİ			•
7	AKŞEHİR KİLECİ MESCİDİ			•
8	KONYA TERCÜMAN MESCİDİ			•
9	KONYA KINACI MESCİDİ		•	
10	KONYA MAHMUD MESCİDİ	•		
11	KONYA AKINCI MESCİDİ	•		
12	KONYA HATUNİYE MESCİDİ		•	
13	KONYA HACI FERRUH MESCİDİ			•
14	KONYA BEGAVİ MESCİDİ	•		
15	KONYA FERHUNİYE MESCİDİ			•
16	KONYA ŞEKERFURUŞ MESCİDİ			•
17	KONYA ERDEMŞAH MESCİDİ			•
18	KONYA HOCA AHMET FAKİH MESCİDİ		•	
19	KONYA ŞİFAHANE MESCİDİ			•
20	AKŞEHİR ALTUNKALEM MESCİDİ			•
21	AKŞEHİR FERRUHŞAH MESCİDİ			•
22	AKSARAY CINCIKLI MESCİD		•	
23	AKŞEHİR GÜDÜK MİNARE MESCİDİ			•
24	ALANYA AKŞEBE SULTAN MESCİDİ			•
25	KONYA HÜSEYİN PAŞA MESCİDİ	•		
26	AKŞEHİR KÜÇÜK AYASOFYA MESCİDİ			•
27	KONYA ZEVLE SULTAN MESCİDİ			•
28	KONYA ALİ HOCA MESCİDİ		•	
29	KONYA TACÜLVEZİR MESCİDİ	•		
30	KARAMAN SAADETTİN ALİBEY MESCİDİ			•
31	AKŞEHİR TAŞ MEDRESE MESCİDİ			•
32	AKŞEHİR HACI HAMZA MESCİDİ			•
33	KONYA HALKABEGÜŞ MESCİDİ			•
34	AKŞEHİR KIZILCA MESCİDİ			•
35	AKŞEHİR KALAYCI MESCİDİ			•
36	KONYA İÇKARAARSLAN MESCİDİ			•
37	KONYA KARATAY MESCİDİ			•
38	KONYA ZENBURİ MESCİDİ			•
39	KONYA KADI İZZEDDİN MESCİDİ		•	
40	KONYA ABDÜLAZİZ MESCİDİ			•
41	KONYA ANBER REİS MESCİDİ	•		
42	AFYON YUKARI PAZAR MESCİDİ			•
43	KONYA ŞEYH SADREDDİN KONEVİ MESCİDİ		•	
44	KONYA ABDULMÜMİN MESCİDİ			•
45	KONYA SIRÇALI MESCİD			•
46	KONYA BEYHEKİM MESCİDİ			•
47	KONYA HOCA HASAN MESCİDİ			•
48	KONYA İNCE MİNARELİ MEDRESE MESCİDİ	•		
49	KONYA SAHİP ATA MESCİDİ			•
50	HARPUT ALACA MESCİD			•
51	KONYA BULGUR DEDE MESCİDİ			•
52	KONYA CEMEL ALİ DEDE MESCİDİ			•
53	KONYA ULVİ SULTAN MESCİDİ	•		

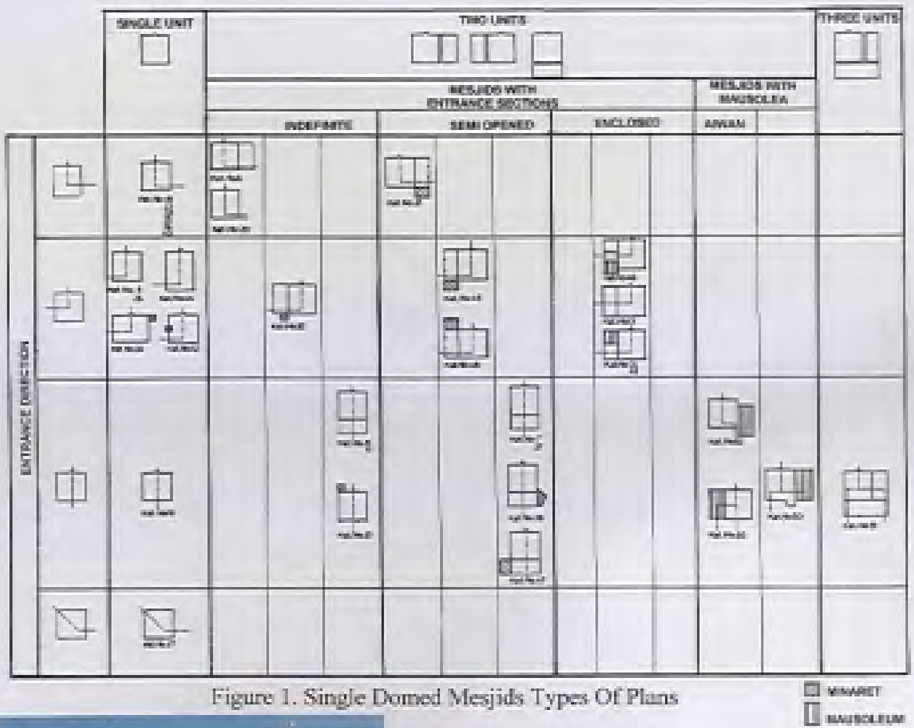

Figure 1. Single Domed Mesjids Types Of Plans

Figure 2. Konya Sırçalı Mesjid (Cat.No:45)

Figure 3. Konya Tercüman Mesjid (Cat.No:8)

Figure 4. Aksehir Tas Medrese Mesjid (Cat.No:31)

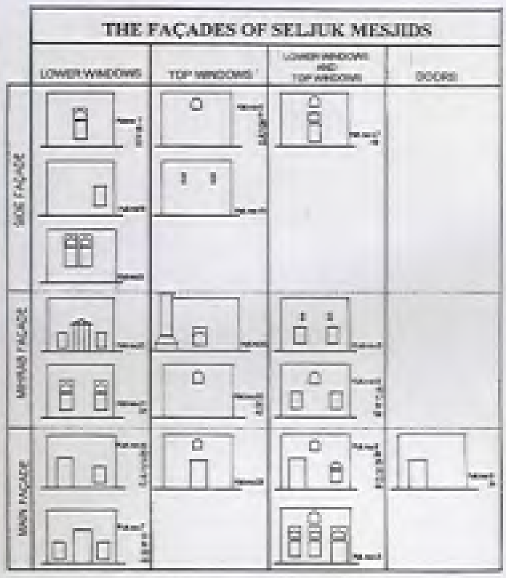

Figure 5. The Façades of Seljuk Mesjids

Figure 6. Konya Hacı Ferruh Mesjid(Cat.No:13), portal

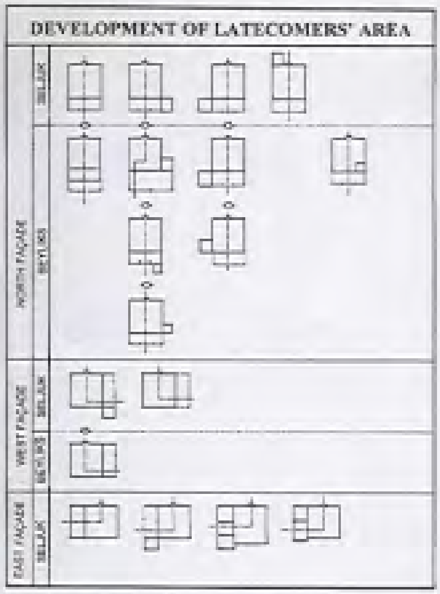

Figure 7. Development of Latecomers' Area

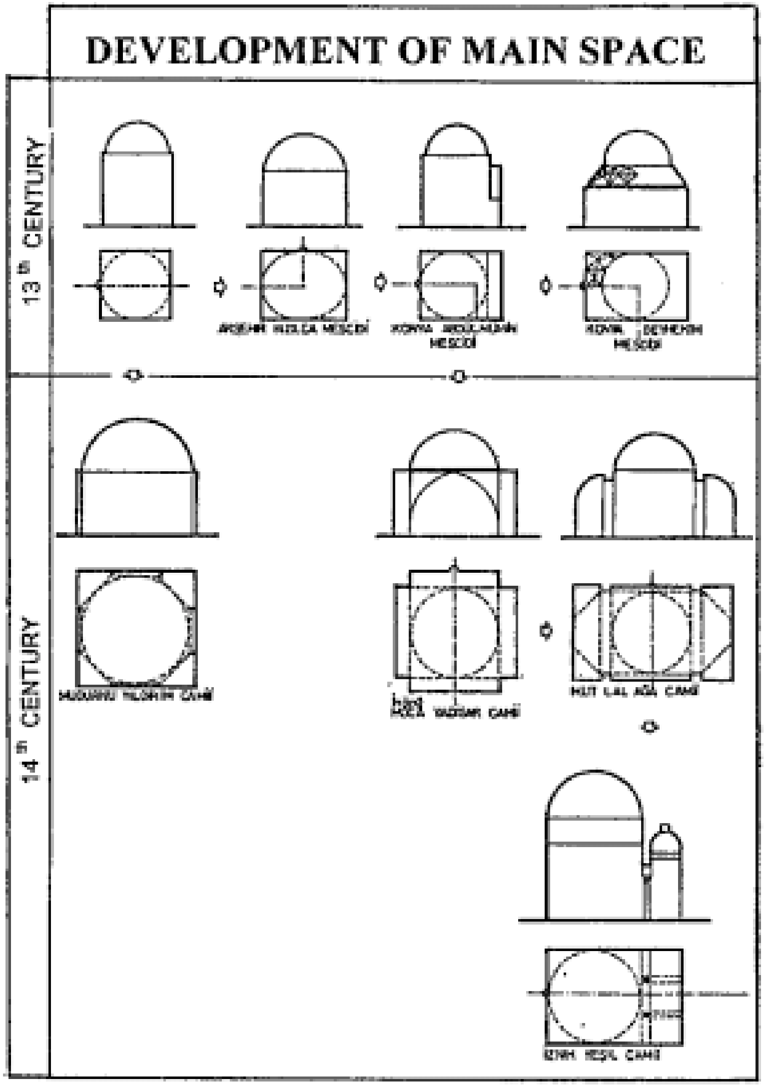

Figure 8. Development of Main Space

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MAKING OF THE JAPANESE TIMBER-FRAMED HOUSES

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ABSTRACT

The Japanese buildings are mainly timber-framed and their structures comprise two types. One is Horitate with posts which are fixed in the ground. The other is Ishizue with posts which are hinged on stone bases. Horitate was seen before the Buddhism introduction in the sixth century, while Ishizue found its way into Japan after the introduction. The Shinto shrine Ise is typical of Horitate, while the Buddhism temple Hohryu-ji is typical of Ishizue. Horitate, contrast to Ishizue, is so primitive, simple but short-lived that the vernacular houses are supposed to have changed into Ishizue from Horitate during the late medieval and the early modern era.

This paper can make the process clear. There should be two changes. One is a change that an existing building of Horitate was pulled down and then a building of Ishizue was newly erected on the same site. The other is a change that an existing building of Horitate remained while some rotted feet of the posts were repaired with Ishizue, that were hinge-ends' post-bases. The former is drastic while the latter is gradual. Particularly in the gradual case hinged-end posts were added to the building without any demolition, though it was necessary to keep some posts to be fixed-ends in the ground. Eventually when all fixed-end's posts in the ground were rotted, the building was not able to be stable.

The paper concludes that the gradual change from Horitate to Ishizue was the major transformation of the Japanese timber-framed buildings especially in terms of the lower class houses.

1. INTRODUCTION

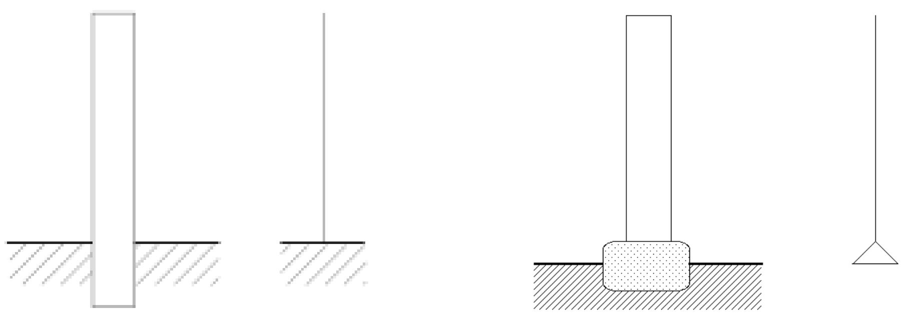
Wooden buildings are generally divided into two structure. One is wall system and the other is skeleton system. The latter is, in other word, timber-framed construction. It is said that at the initial stage of wooden buildings' history straight timber such as coniferous tree was suitable for wall system while winding timber such as broad-leaved tree was suitable for skeleton system. Wall system could be seen in the northern parts of Europe where coniferous tree was available while skeleton system could be seen in the middle and southern parts of Europe where coniferous tree was available. In case of Japan, there were both coniferous and broad leaved tree, but the wooden skeleton system covered Japan. It means that the main Japanese wooden buildings was not wall system but timber-framed. How had they been made ?

2. VARIATIONS OF STRUCTURE

The three variations of structure are briefly described below to understand the making of the timber-framed construction in Japan . The first variation is post-bases, the second is timber-frame and the third is roof-truss.

2.1. VARIATIONS OF POST-BASES; FIXED END OR HINGED END

Japanese timber-framed buildings are skeleton system and it comprises two types of post-bases (figure 1). First is with fixed ends at post-bases (figure 1-1). Second is with hinged ends at post-bases (figure 1-2). The fixed end's post-base is called Horitate which means to dig and to erect, while the hinged end's post-base is called Ishizue which means stone bases.



Horitate fixed end — figure 1-1

Ishizue hinged end — figure 1-2

figure 1. Variations of post-bases; fixed end or hinged end

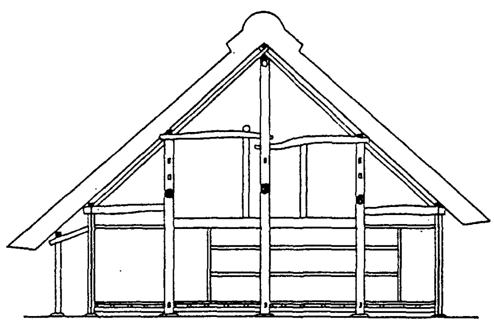
In case of Horitate posts were erected in pits and then bottoms of the posts were fixed (figure 1-1), while in case of Ishizue posts were erected on stone bases which were put on the ground (figure 1-2). The Horitate structure could be seen in vernacular houses as well as in the ancient shrine Ise, while the Ishizue structure could be seen in upper class buildings as well as the ancient temple Horyu-ji. The hinged end of Ishizue appeared just after the introduction of Buddhism into Japan from the continent. On the other hand the fixed end of Horitate did exist before the introduction. Some upper buildings and almost all of religious architecture were built on the hinged ends' post-bases of Ishizue. Yet, the shrine of Ise has never changed its style with the fixed ends' post-bases of Horitate by making its copy every twenty years for fourteen hundred years. In addition most of vernacular houses kept its style with fixed ends' post-bases of Horitate.

It is during the late medieval and the early modern era that the vernacular houses changed to be erected on the hinged ends' post-bases of Ishizue. The way how the change happened should be described.

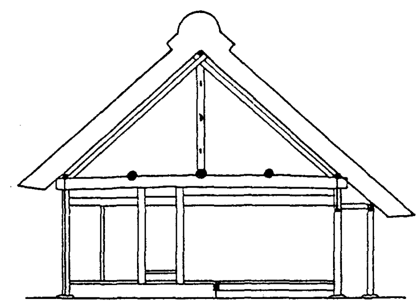
2.2. VARIATIONS OF TIMBER-FRAME

As mentioned above, the post-bases of the Japanese historical buildings comprise Horitate and Ishizue, and their superstructure also comprise two types according to the post-bases (figure 2).

First is a building which is not divided into a frame and a roof-truss. Second is a building which is divided into them. In the first case a ridge is supported by a middle post on the ground in a gable end (figure 2-1). The post is called Munamochi-bashira in Japanese. On the other hand in the second case a ridge is supported by a middle post on a tie beam in a gable end (figure 2-2). The post can be called Odachi or Muna-zuka in Japanese.



An example of no divided superstructure
figure 2-1



An example of divided superstructure
figure 2-2

figure 2. Variations of superstructure

It has already proved that most of the historical buildings in Japan had hinged ends at the post-bases and the skeletons were divided between a frame and a roof-truss. On the other hand some archaeological documents as well as some historical pictures suggest that the pre-early modern buildings had fixed ends' post-bases and were not divided between a frame and a roof-truss.

It has been concluded that the Japanese vernacular houses changed into the structure both with the hinged-ends' post-bases and with the division between a frame and a roof-truss, from the structure both with the fixed-ends' post-bases and with no division between a frame and a roof-truss. Main topics is the way how the change happened.

2.3. VARIATIONS OF ROOF-TRUSS

(1) APEX DETAILS

Japanese historical buildings comprise three types of apex details (figure 3). The first and the second has one ridge, while the third has two ridges.

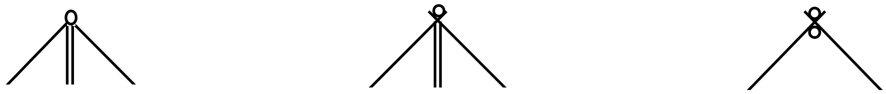


figure 3-1 figure 3-2 figure 3-3

figure 3

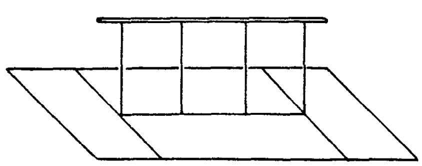
In the first type, a middle post supports a ridge, and principal rafters are set on the both sides of the ridge (figure 3-1). It can be seen mainly around the capital city Kyoto. In the second a post supports an intersection of principal rafters and a ridge is put on the intersection (figure 3-2). It can be seen in the peripheries of Japan. In the third type a middle post supports a lower ridge, an intersection of principal rafters are put on the lower ridge and a upper ridge is put on the intersection (figure 3-3). It can be seen in the shrine of Ise and in some vernacular houses.

The first and second type are simple and major, but the third type is upgraded and minor. So the first and second one should be dealt.

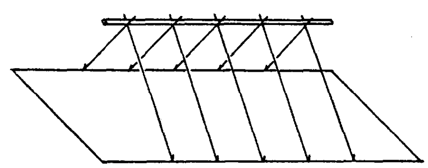
(2) ROOF-TRUSS

Roof-truss is mainly composed of two types of Odachi and Sasu, depending on the two types of apex details (figure 3-1 & 3-2) just mentioned above. The Odachi structure has posts which support a ridge directly from tie beams (figure 4-1). The post is called Odachi, which is similar to a king post in English timber-framed buildings. While the Sasu structure has two intersected rafters which can supports a ridge (figure 4-2 & 5) and sometimes has posts between the intersections and tie beams (figure 2-2 & 9-2). The intersected principal rafters are called Sasu. The posts between the intersections and the tie beams can make the skeleton stronger, but they does not necessarily required to keep the skeleton stable.

It has been proved that firstly the Sasu structure came into being during the late sixteenth century and the early seventeenth century, secondly it appeared later than the Odachi structure and thirdly it located in the different area which the Odachi structure covered.



Odachi structure
Figure 4-1



Sasu structure
figure 4-2

figure 4. Roof-truss

(3) POST UNDER RIDGE

In case of the Odachi structure a middle post on a tie beam is called Odachi. Historical archives suggest that the original term of Odachi meant a middle post which was erected in a pit on the ground and supported a ridge directly from the earth. It can be said that the middle post on the ground in the gable end is an original meaning of Odachi. Later the post on the ground changed into a post on a tie beam. In other word, the old is the skeleton with no tie beam, while the new is the skeleton with tie beams. In addition most of the skeleton with no tie beam depended on the fix ends' post-bases (figure 6, 7, 8-1 & 9-1). While most of the skeleton with tie beams depend on the hinged ends' post-bases under the condition that posts are jointed with tie beams rigidly (figure 8-2, 9-2, 10 & 11).

On the other hand in case of the Sasu structure a post between the intersection and the tie beam does not necessarily required to keep the skeleton stable. The Sasu structure with no post between them is as stable as the Sasu structure with a post between them. The reason why the post between them can be seen in some of the Sasu structure at times. It depends on the social backgrounds.

As mentioned above in the Odachi structure the post on the tie beam came from a post on the ground. It can also be said that in the Sasu structure the post on the tie beam came from the post on the ground as similarly as the Odachi structure did. Then the change must be described clearly.

3. WAYS OF CHANGES

Some ways of changes are indicated below in order to explain the dynamics of the Japanese timber-framed houses during the late medieval and the early modern era.

3.1. CHANGE OF POST-BASE

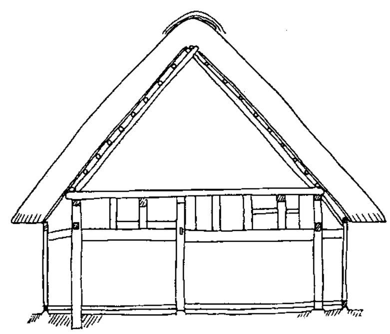
The change of post-bases is the most important matters in terms of the change from the old skeleton to the new skeleton.

Most of the old skeletons had no division between a frame and a roof-truss, while most of the new skeletons had division between a frame and a roof-truss. In additon the old skeletons were erected on fixed ends' post-bases, while the new skeletons were erected on hinged ends' post-bases. These two contrasts indicate that there were two changes between the old skeletons and the new skeletons. The first change is from the skeleton with no division between a frame and a roof-truss into the skeleton with division between them. The second change is from the fixed ends' post-bases to the hinged ends' post-bases. The first change depended on the second change. The change into the hinged ends' post-bases caused the division between a frame and a roof-truss. In this meaning the most important events was the change of post-bases from the fixed end of Horitate into the hinged end of Ishizue.

One more thing have to add to the explanation of the post-base's change. There were two different types in terms of the post-base's change. First type is a simple and drastic change. It linked to the Horitate structure in which each post is stuck into the ground. In this case the bottom of posts were easily rotted. If all posts were rotted and the construction was not stable, the building should have been pulled down and then the new building was erected on the hinged end of every post-base. The new posts on the stone base could keep longer than the old posts on the ground. This change is understandable.

On the other hand the second type is a complex and gradual change. It also linked the Horitate structure. If a bottom of a post was rotted, only the rotted part was cut off, a stone base is inserted on the ground and then a new part is put into the bottom of the post. In this type bottoms of posts gradually changed into stone bases on the ground. This change made a post-base hinged. So it is impossible to change every fixed end's post-base into hinged end's post-base. It is because the structure could not be stable if all post-bases were hinged ends without any rigid joints between posts and beams. Yet, if only a post-base kept a fixed end at a gable end, the structure is still stable. Therefore it can be said that the gradual change from the fixed end of Horitate into the hinged end of Ishizue deserves more attention.

Actually there is a good example. The house of Yamada Yoshinori in Akiyama-go in the northern part of Nagano (figure 5) suggests the complex and gradual change from Horitate into Ishizue. The house is located in the mountain district. It is not only located in the out-of-the-way place but seems to be very old. It is the interesting things that the house has both fixed ends' posts and hinged ends' post. The post A in the figure 5 has a fixed end, while the post B and C have hinged ends. The house has one more fixed end's post-base. The house is an typical example which shows the second type of change from Horitate into Ishizue.



A B C

figure 5. Yamada Yoshinori's house in Nagano

3.2. STRUCTURAL STABILITY

Then, the structural stability has to be examined. The formula (1) and (2) judge whether the structure is stable or instable.

k: number of joint of framework
s: number of member of framework
r: number of member of rigid joint
n: number of bearing capacity
(roller bearing 1, pin bearing 2, fix end bearing 3, per a supporting point)

Instability; $2k > (n+s+r)$
Stability 1; staticallydeterminate $2k = (n+s+r)$ (1)
Stability 2; statically indeterminate $2k < (n+s+r)$
In the case of statically indeterminate structure;

(n+s+r)-2k=m (2)

The structure is 'm' dimension(s)' statically indeterminate one.

The two skeletons of different size have to be checked. The first skeleton has two spans at a gable end (figure 6). The second skeleton has four spans at a gable end (figure 7).

Firstly the skeleton of the figure 6-1 has six joints of framework (k=6), five members of framework (s=5), no member of rigid joint (r=0) and three fixed ends which means nine bearing capacity (n=3*3=9). The skeleton is proved to be stable and statically indeterminate (2k<n+s+r, m=2). On the other hand the skeleton of the figure 6-2 has six joints of framework (k=6), five members of framework (s=5), no member of rigid joint (r=0) and one fixed end and two pins which means seven bearing capacity (n=3*1+2*2=7). The skeleton is proved to be stable and statically determinate (2k = n+s+r, m=0). This means that the change from fixed end to hinged end could keep the structure stable in the condition that one post did keep to be a fixed end at the bottom of in the skeleton.

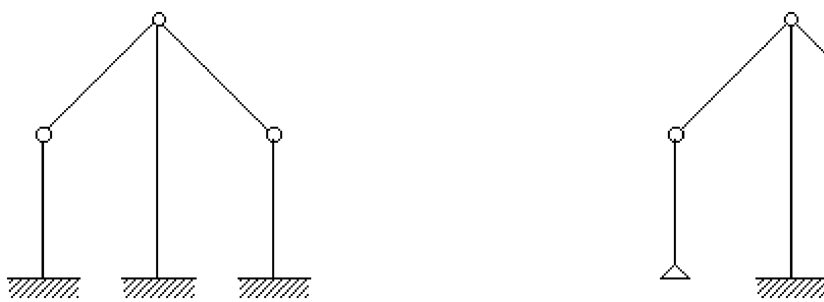


figure 6-1 figure 6-2

figure 6. smaller skeleton of two spans at a gable end

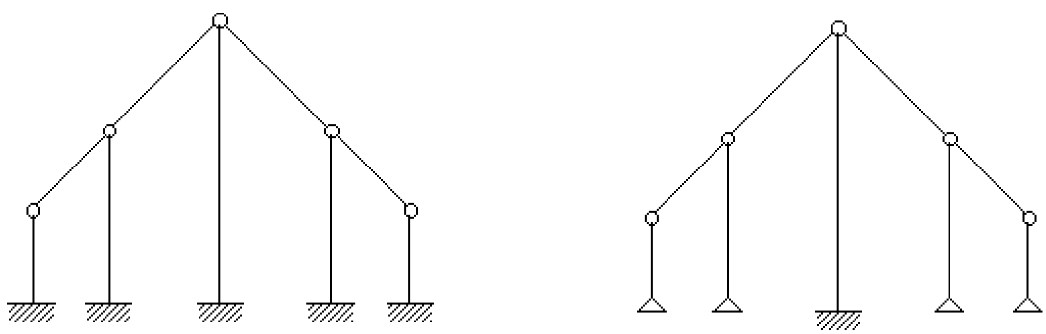


figure 7-1 figure 7-2

figure 7. bigger skeleton of four spans at a gable end

Secondly the skeleton of the figure 7-1 has ten joints of framework (k=10), nine members of framework (s=9), no member of rigid joint (r=0) and five fixed ends which means fifteen bearing capacity (n=3*5=15). The skeleton is proved to be stable and statically indeterminate (2k<n+s+r, m=4). While the skeleton of the figure 7-2 has ten joints of framework (k=10), nine members of framework (s=9), no member of rigid joint (r=0) and only one fixed end and four pins which means eleven bearing capacity (n=3*1+2*4=11). The skeleton is proved to be still stable and statically determinate (2k=n+s+r, m=0). This means that the change from the fixed end to the hinged end could keep the skeleton stable under the condition that one post did keep to be a fixed end at the bottom of in the skeleton, as similarily as the skeleton of figure 6.

3.3. CHANGE FROM FIXED END INTO HINGED END

The figure 6 depicts the smaller skeleton of two spans at the gable end. Both the figure 8 and the figure 9 depict the structural transformations of the skeleton of the figure 6. The figure 8 has the apex detail of the figure 3-1, while the figure 9 has the apex detail of the figure 3-2.

Firstly the skeleton of the figure 8-1 shows that a post supports a ridge directly. The skeleton of the figure 8-2 shows a modernised skeleton of the figure 8-1 after the change into the hinged end of Ishizue from the fixed end of Horitate. The skeleton of the figure 8-2 comprises a frame and a roof-truss in which the post on the tie beam supports the ridge directly.

Secondly the skeleton of the figure 9-1 shows that a post supports a ridge indirectly via an intersection of principal rafters. The skeleton of the figure 9-2 shows a modernised skeleton of the figure 9-1 and it comprised a frame and a roof-truss in which the post on the tie beam supports the ridge via the intersection of principal rafters.

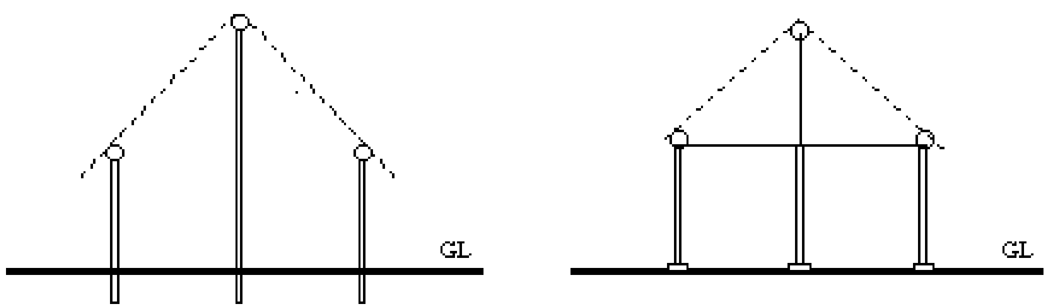


figure 8-1 figure 8-2

figure 8. change into Odachi

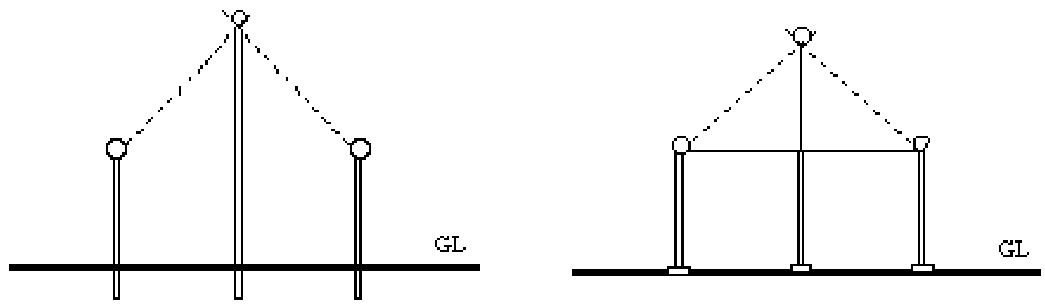


figure 9-1 figure 9-2
figure 9. change into Sasu

Then what is the difference between the skeletons of the figure 8-2 and the figure 9-2 ? The skeleton of the figure 10 is one of the most modernised structure of the Japanese vernacular houses. As mentioned above there were two moderniesed skeletons (the figure 8-2 and figure 9-2) and the skeleton of the figure 10 is the most modernised one.

The skeleton of the figure 8-2 could not change into the skeleton of the figure 10 without any difficulty, because the ridge had to be supported by the post on the tie beam directly. In contrast of the figure 8-2, the skeleton of the figure 9-2 could change into the figure 10 easily, because the ridge was able to be supported by the intersection of the two principal rafters without any supports by the post between the intersection and the tie beam.

In this meaning the skeleton of the figure 10 came from the skeleton of the figure 9-1 via the skeleton of the figure 9-2. While the skeleton of the figure 8-1 could change into the skeleton of the figure 8-2, but not change into the skeleton of the figure 10 without any difficulties.

It can be concluded that the most modernised skeleton of the figure 10 came from one of the two fixed end’s structure. Exactly the figure 10 came from the figure 9-1.

The second bigger skeleton of four spans (figure 7d) also changed into the modernised structure which had the hinged ends' post-bases and the division between a frame and a roof-truss (figure 11) as well as the smaller skeleton of two spans.

It can be said that the post between the ridge and the tie beam (figure 8-2 & 9-2) came from the post on the ground (figure 8-1 & 9-1) not only in case of the smaller skeleton but in case of the bigger skeleton. Later the post of figure 9-2 was able to disappear and to change into the skeleton of figure 10, while the post of figure 8-2 was not able to disappear.

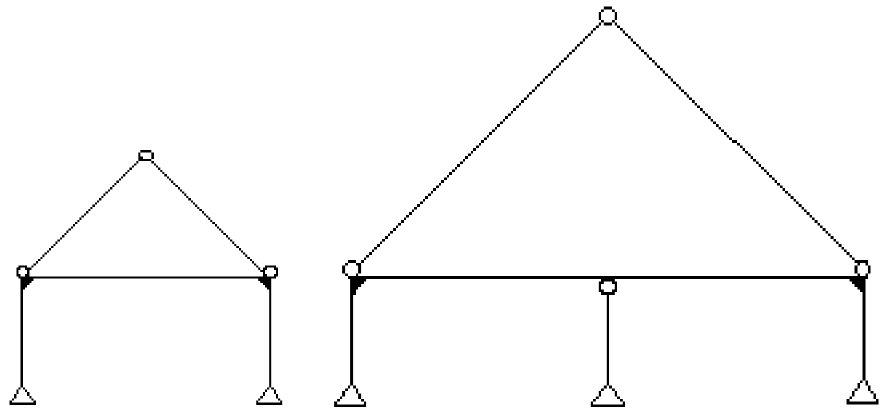


figure 10 figure 11

4. SOCIAL BACKGROUNDS

Now, it has to be pointed out that the post on the tie beam could be seen much more in the second skeleton of four spans than in the first skeleton of two spans. The small skeleton belonged to the lower class within a society at that time, while the bigger skeleton belonged to the upper class within a society at that time. In addition the smaller skeleton existed much more than the bigger one did.

Then why could the post on the tie beam be seen much more in the bigger skeleton than in the smaller one ? It is because the bigger skeleton, which belonged to the upper class, could change into the hinged ends' post-bases of Ishizue from the fixed ends' post-bases much earlier than the smaller skeleton, which belonged to the lower class. The smaller skeleton was obliged to remain the old structure of the fixed ends' post-bases, but later in the post-medieval era it could change into the modernised structure such as the figure 8-2 and 9-2. Above all some of the smaller skeletons of the figure 9-1 could change directly into the most modernised skeletons of the figure 10 or 11 which had no post between ridges and tie beams.

In other word the smaller skeletons of two spans (figure 6, 8 & 9) had to keep the structure with fixed ends' post-bases for much longer time than the bigger skeleton, but later it could change into the modernised skeletons, one of which could changed into the most modernised one with no post between a ridge and a tie beam (figure 10 & 11). On the other hand the bigger skeletons could change into the structure with the hinged ends' post-bases much earlier than the smaller ones, because the bigger skeletons were managed by the richer people of the society at that time.

5. CONCLUSIONS

Just after the first stage of the introduction of Buddhism into Japan at the sixth century the hinged ends' post-bases could be seen in the religious architecture such as Horyu-ji. Yet, most of Japanese timber framed construction originally had the fixed ends' post-bases of Horitate, which were stuck into the ground. The structure of Horitate could be seen in vernacular houses of the early modern era as well as in Ise. Horitate changed into hinged ends' post-bases of Ishizue during the late medieval and the early modern era especially in terms of vernacular houses in Japan

There were two types of change from Horitate into Ishizue. One was simple and drastic change which happened mainly in the upper class's bigger houses, while the other was complex and gradual change which happened mainly in the lower class's smaller houses. In case of the latter change, the rotted part of the post was cut off, a stone was put on a base and a new part was put into the rotted part when the bottom of a post was rotted. The change was the major transformation of the Japanese timber-framed buildings.

Just after the change into Ishizue from Horitate, the skeleton was divided between a frame and a roof-truss. There were two types of roof-trusses depending on the two types of apex details. One was the Odachi structure and the other was the Sasu structure. The both came from the old structure with middle posts which were stuck into the ground and supported the ridge from the ground. One of these skeletons could change into the most modernised structure, which is composed of a frame and a roof-truss with hinged ends' post-bases but no post between a ridge and a tie beam (figure 10 & 11).

It is the major change from Horitate to Ishizue that modernised the Japanese timber-framed houses. The change caused the division between a frame and a roof-truss, most of which can be seen in the present Japanese traditional houses.

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THE STRUCTURAL EVALUATION AND REINSTITUTION OF THE 17TH CENTURY OTTOMAN WARSHIP CLASS "BASTARD"

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ABSTRACT

Galley is a vessel of Mediterranean origin.It has been used untill the beginning of the 19 th century, because it confirmed the required needs of the region , fitting the changing conditions in the Mediterranien world.Galley is avessel which sails by the use of oars and if need it can also move by the use of sails.It has generally been use in battles for thousands of years, so it didn't change much.The most important change in the galley was the elevation of the rostrum from the sea level up to thelevel of the deck.Another change became after the use of the firearms.İt was made by joining the gunport to the bowhead.This type of vessel was used between the 14-19 th centuries in the Mediterranean region and was accepted as the most valuable ship of the navy.As it was used by the other nations of the Mediterranean.this vessel was the main war ship of the Ottoman Navy,gaining a famous name.Altough designed for the same purpose, the Ottoman Galley had different specialities compared with the other galleys of the Mediterranean.Unfortunately none of the Ottoman galleys have reached our day.No models and tachnical drawings exist. The only sources are miniatures,engrawings,manuscripts and the historical texts.Being sufficient these sources help to us to discover the prototyp of the galley,that was used in the middle of the 17 th century.The main aim of this work is to study the structure,construction model and the materials used,afterwards making a reinstitution of the 17 th century prototype of the Ottoman war galley.

1. GALLEYS AND THE "BASTARD"

Galleys were used for war and trade throughout the recorded history of the Mediterranean.It is very suprising that a ship class was able to continiue its existence in about more than 3000 years'period.However,the succes of the galleys in adapting to the changes of the time and the compatibility of the Mediterranean region with the special conditions enabled them to continue their existence until the 19th century.Since the windless periods that lasted long especially in summertime caused only the ships with big tonnages to become motionless,the ships moving by rowing always constituted an important alternative.A ship moving by roving had the ability to move continuously even if it moved at a slow speed.When the war was in question,this situation made the galleys advantegeous in the battlefield until the 17th century.Another reason for this was the building of the galleons for a multi-purpose usage without considering whether they were war ships or trading ships until the 17th century,and the numbers of their cannons being fewer and especially their being with short range when compared to the galeons of the next years.However,the galleys could be equipped with cannons with wery long ranges since they were long,and the cannons could be located in the direction of bow and stern.In this situation,what was necessary for a galley to be able to fight against a galleon that was equipped better was to catch it in a windless sea by following it,and by keeping out of range,to make it surrender after making it unable to fight with its cannons.However,if the enemy had been wery strong,the groups consisting of more than one galley would have been used.On the other hand,if the winds had been suitable for the galeon to move very quickly,it would have become advantageous,and by moving towards the weakly structured galleys,the galleon would have rammed the galleys that could not move as quickly as itself.In such a situation,of course,galley or the group consisting of galleys would have preferred to move away from this situation,which was not advantageous for them.Galleys were originated in the Mediterranien,and they were not used much in the other seas of the world.In the Ottoman history,the number of the military cruises in the oceans in which the galleys participated was wery small.In addition,most of these military cruises resulted in failure.Among these were the cruises to America and South Africa.These vehicles were insufficent to cope with the storms of the high seas with their qualifications such as their fragile structures,length and width rates like 1-10,low board hights and whole staff’s traveling in open air.Therefore,the galleys were very rarely used outside the Mediterranean Region by the Mediterranean nations.Since the Turkish groups that founded the Ottoman State did not have any knowledge concerning naval experience before they come to Anatolia,it is accepted that Turkish navigation began with the conquest of Anatolia.Therefore,when the Turks settled down in Anatolia,it became necessary for them to have an intrest in the navigation.First,they made use of the experience and knowledge of the naval cultures they got into touch with.The first relationship and interaction were with Byzantine.Some of the historians argue that the word **‘kadırga’** (Galley) in Turkish language was the transcription of the word **‘Kaderga’** which was the name of a type of a ship in the Byzantine navy **[1].**All the other Mediterranean nations used the galley both as a warship,and as a trading ship.It is accepted by the authorities that the galleys did not change much in general from antique age to the fiery guns’period.The basic change is said to be the rising of the rostrum,which had been at the water level to the deck.Moreover,it became necessary to allocate a place for the heavy fiery guns on the ship.However,when examined more carefully,it can easily be realized that there were important differences between an antique age galley and a 17^{th} -century-galley.Not only between the ships of different periods but also between the ships of the same time period owned by different nations,important differencies can be observed.The first Ottoman galleys hawing the visual descriptions belonged to the end of 15^{th} century,and the last ones belonged to the end of 18^{th} century **[2].**From these descriptions,about ten galley classes can be recognized.If it is taken into cosideration that the documents are arbitrary and there is no effort to authenticate,we can consider that much more class ships could have been made during the period in which the galleys were used in the Ottoman navy.If other ships moving by rowing,which were the derivatives of the galleys,are cosidered,it can be said that this number can easily exceed forty.This class was the most prestigious ship class of the Ottoman Navy between the 15^{th} and 17^{th} centuries.According to the records of the Imperial Shipyard (The old Imperial Shipyard located in the north of golden horn),between the years 1610 -1663, among the ship classes moving by rowing 71 bastard (baştarda) **(Figure-1) ,**were made and 102 were repaired, 180 galleys (kadırga)

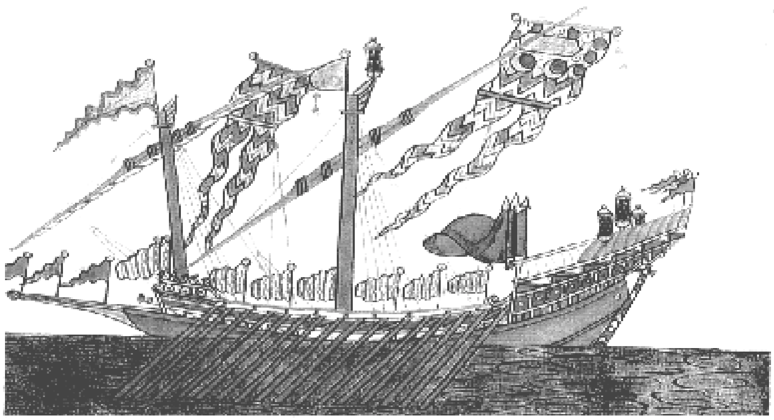
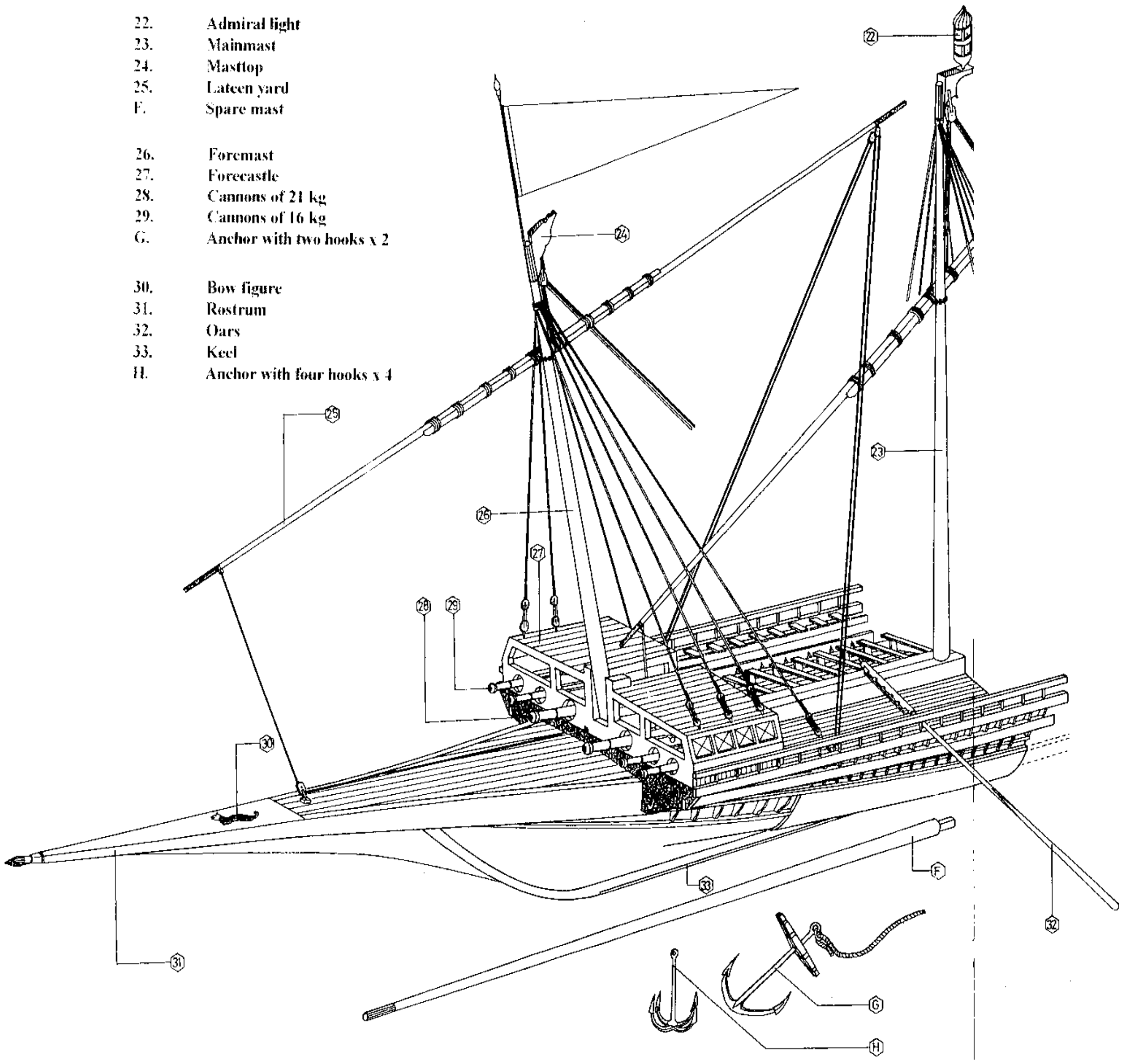


Figure 1

were made and 413 were repaired,and 16 mahone (mavna) and 35 galliots (kalite) were made.**[3]** If we consider that Imperial Shipyard was an organisation with the aim of not building but equipping,and the Empire had more than 20 shipyards and the main building activities were made in these shipyards,the possibility that the real production number could be higher is getting stronger.In the next years,the boats moving by rowing went back to the secondary position with the developments in galleons,and later on they disappeared completely.In this study,the first rate war galley,which will be analyzed,was a ship class moving by rowing and being used in the first half of the 17^{th} century in the Ottoman Navy.Although these kinds of ships could also move with sails,they had the ability to use the wind efficiently only when it got the wind from certain directions.They could not veer towards the wind blowing from the opposite directions.One of the two reasons why the first half of the 17th century have been chosen is that there are sufficient written and visual documents in both Ottoman and foreign sources.The other reason is that the ships of the time were more successful in the aspect of aesthetics than the ships of the previous and next periods.In addition,in the inventory of the Turkish Naval Museum,there is a big Sultanate caique with “galliot” (24 seated) forms that belongs to this time.Its basic structure is very similar to galley.This makes the analysis easier.

Basic ship classes moving by rowing (they also have sails) in the Ottoman Navy were as follows:

1.	**Warship with 10-17 seats (banks)**	**– two or three men row each oar.**	**Frigate1-3 cannons**
2.	**Warship with 18-19 seats**	**- two or three men row each oar.**	**Pergende 1-3 cannons**
3.	**Warship with 19-24 seats**	**- two or three men row each oar.**	**Quality 2-3 cannons**
4.	**Warship with 25 seats**	**- four men row each oar.**	**Galley 3 cannons**
5.	**Warship with 26-36 seats**	**- five or seven men row each oar.**	**Bastard 4-7 cannons**
6.	**Warship with 26 seats**	**- seven men row each oar.**	**Mahone 24 cannons**



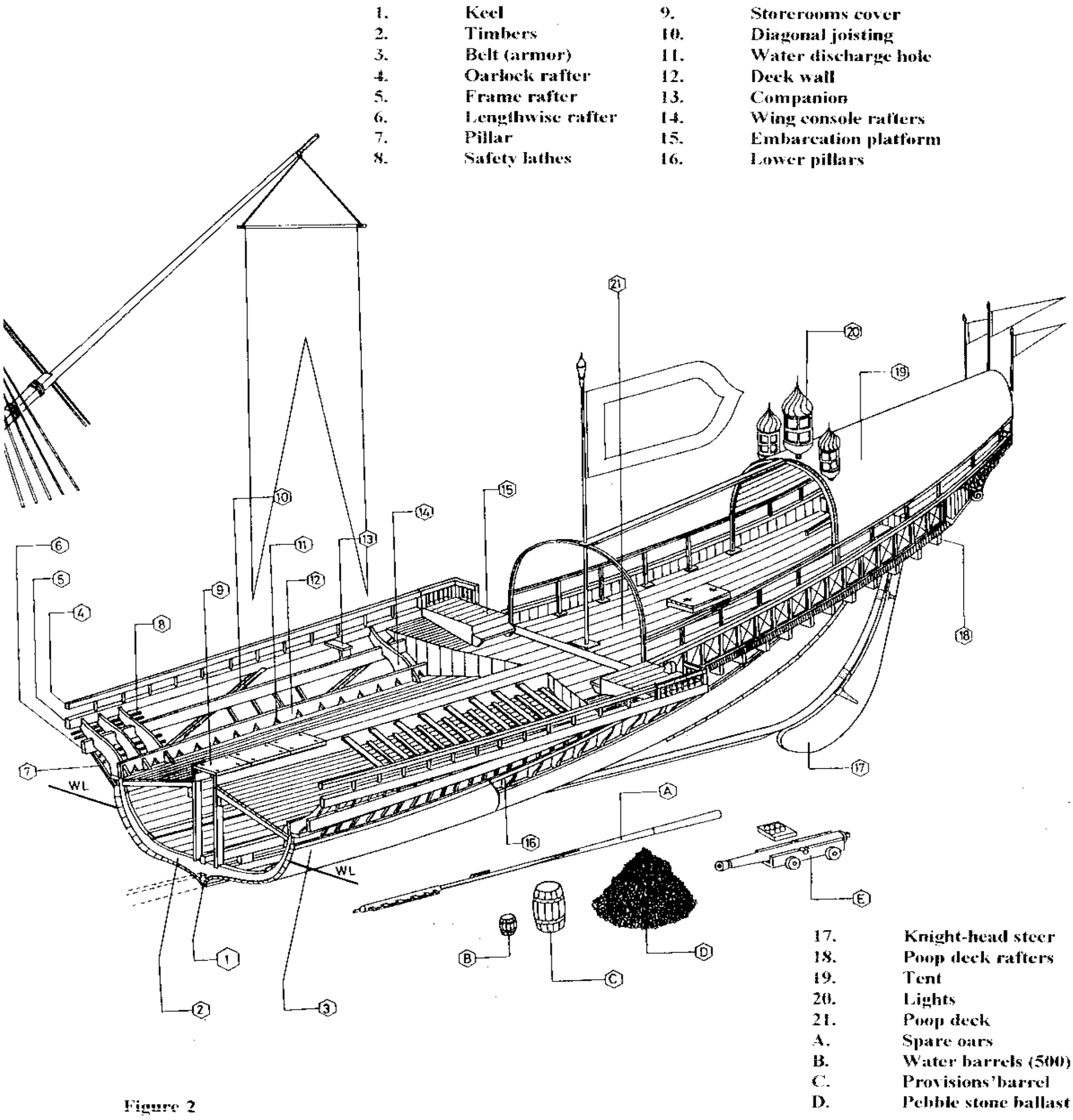


Figure 2

The length of the ships given in the Ottoman sources were keel lenght.That is to say,it was the length of the straight part of the keel which was laid on the dockyard,and which was between the sternpost and the stempost of the ship.In order to figure out the actual length of the ship,the length of the stempost and the sternpost must be added to this length.

Galleys and its derivatives,which were small in size and which had a length-width ratio of 1-9,10 and a small portion under water,consisted of a rectangular deck located on this body.When galleys and its derivatives were in question ,the more seats there were,that is to say,the longer ship was,the bigger the wing console extending from the board towards the sea was**.(Figure-2)**Moreower,while rowing at the same time,the lateral force applied to the rafter carrying the wing console reached at the highest values.The bastard which will be analyzed,was afirst class warship with 30 seats.It was used only by the navy.30 rows of seats meant60 rows of oarsmen on both sides of the symmetry axis.Since one row on the right side was allocated for the fire place,the ship contained a total of 59 rows of oarsmen.As each row contained 5 oarsmen the total number of oarsmen was 295.The warriors sat on the benches that were located in the outermost part,betweenthe oars on the wings.On these 60 benches ,which were called "**manka"** there were 120 warriors;each bench allowing 2 warriors to sit.The number of the stuff in the ship was about 460, including 1skipper,25 sailors,1master sailor,3 errand boys under the command of the master sailor,2 master steersmen,3 oar makers,3 caulkers,3carpenters **[4].** The total length of the ship was approximately 80 meters , including the bow ram. On the other hand , we know that those with 36 seats could have 800 people on it.Moreower,we know that during the following periods,the prototypes with 40 or even 45 seats were built**[5].**The length of a ship with 36 seats could approach about 90 meters.

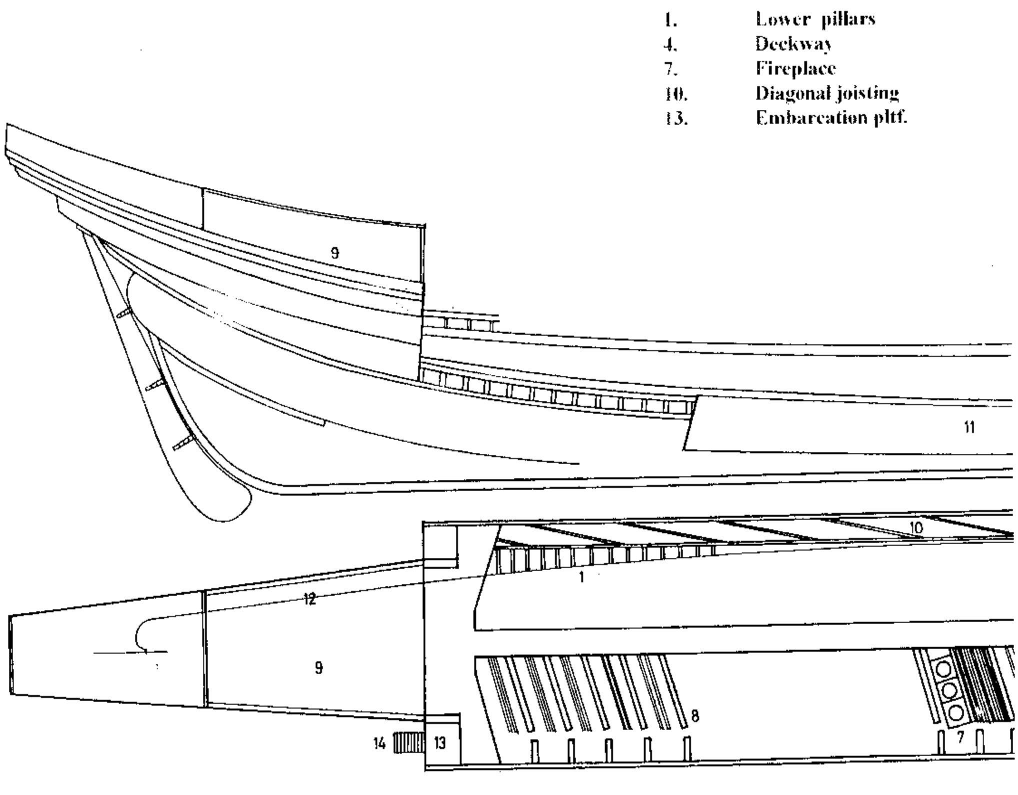
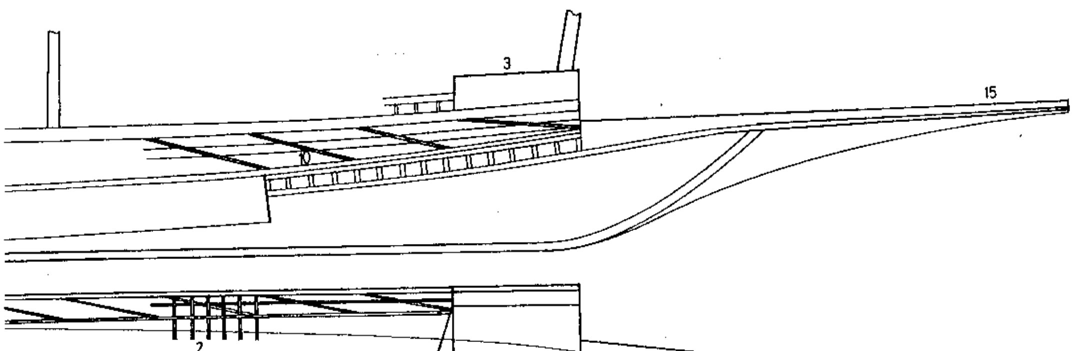


Figure 3

2.STRUCTURAL ANALYSIS AND REINSTITUTION

The main problem in the construction of the structer of a bastard was the fact that it had a very fragile body type.Due to the fact that it was long,it could sit onto two or three waves at the same time**.**This meant that when it was stormy,the ship could be split into peaces.Therefore,the ship had to have a very strong structure.The amount of the nails used in order to gather the wooden parts of the ship was 22 tons.The weight of the cannons in the forecastle was about 16 tons**[6].**Therefore,the ship had to have a very strong keel.Moreower,it probably had to have a secondary internal keel on the frame timbers just like the other big ships of the time.In addition to this,there had to be extra lengthwise rafters,which connect the frame timbers with each other.The wooden armor plate,which was called belt,supported the body against stress and spraining at the same time.In fact,this layer was made to protect the water portion of the ship body from the damages that was caused by heavy guns,because during the period of the sails,one of the most effective war techniques was to pierce the enemy ship at the level of

2.	Wing console rafters	3.	Forecastle
5.	Main mast	6.	Storerooms cover
8.	Seat s	9.	Poop deck
11.	Belt (armor)	12.	Main body line
14.	Ladder	15.	Rostrum



the water portion.As it can be seen in the widthwise section,the deck was low,and inclined outwards,but almost positioned upright on both sides of the platform located in the middle.The reason was that most of the time all,and somethimes,most of the oarsmen were the galley slaves.Therefore,the galley slaves who were chained to the benches had to satisfy all their needs like sleeping,eating at where they were.Poop deck did what a bridge ia a modern sea vehicle did.One of the fundamental engineering problems was to settle a rectangular platform on the oval-shaped body,and to transmit the momentum,which was generated by rowing,to the body of the ship as adriving force.This rectangular frame was settled on the body at three stages.The first stage was a piece of covering connected to the body and that was suitable to the stern,which was a greater section than the coverings.The second was a lengthwise rafter carrying rectangular frame which was carried by the rafters that were linear in the horizontal projection,but curvilinear in the lateral projection(curvilinear in one direction and on one side) and by the ones standing on the first covering piece**.(Figure-3)**When we approach to the bow and stern,naturally,the distance from the body gets longer,and the length of the rafters that carries it also gets longer.The basic rectangular frame was carried by the widthwise rafters in great numbers and has special forms which carried themselves and rose from the body which was connected to this lengthwise rafter.The number of these rafters was over 60 on each side.In addition,there were over 60 narrow crosscut and short buttresses that attack the lengthwise rafter to the belt.And also,in a different approach,the rectangular frame was attached to the curvilinear lengthwise rafter with big-cut rafters about 3-3.5 meters tall to prevent the big momentum,which would exist on the wing during the rowing from damaging the structure**.**With this structure,the Ottoman bastard showed big differencies from the similar ships of other contemporary Mediterranean nations.One of the main reasons of this was the insistence on a navy that consisted of ships moving by rowing against their enemies that gradually began to use galleons as main warships.The reason for this is explained in old texts as follows: “The warships should move quickly and should not be dependent on the wind.The navy should be able to go whenever and wherever it wants to”. It was probably for this reason that the main warship class continued to be the galleys for a very long time,and the ships having quite different structures when compared to the similar ships owned by other contemporary nations appeared.The ship,like its contemporary ships,had two masts that could be dismantled when it was necessary.It had two big lateen yard carrying the triangular lateen sails that was attached to these masts.When necessary,rectangular sails (square-rigging) could also be attached to these lateen yards.The sails were round sails having really big area and they could be taken with the mast during the wartime.During the cruise,30 spare oars and spare lateen yard were carried in the store of the ship or being attached to the lower part of the wing.The ship had two anchors with two hooks and four anchor with four hooks.Its bottom were made extremely smooth to enable it to move quickly,and the part of the body which was under the water was oiled after it was covered with pitch twice,before the cruise and during the cruise lasts four months.In this way,the body became smooth and water resistance decreased to the lower degree.It has been stated in the sources that a ship of this kind could move at over 12 miles per an hour under suitable wind conditions.However,if the ship had had to move by rowing for a long time,such a big ship would have had to use half of the benches in order to get its rowers to rest,it would only move at a speed of 3 or 4 miles per an hour.Although the ship was not suitable for the high seas,in summer months,it had a suitable structure to be used in the Mediterranean,which is an inland sea.The extraordinary sheerline of the poop deck enabled the the eye levels of the steersman and the skipper of the ship to be higher than the highest point of the forecastle; that is to say,it enabled them to see the front part of the ship easily.This brought an advantage while sailing the ship.These ships were pulled out of the sea onto their covered stocks and they spent the winter in these places called cell.Without any extraordinary conditions,it was impossible for these ships to put out to the sea.However,in the next periods,the galleys had to put out to the seas with the galeons.They left the battlefield completely at the end of the 18th century**.[7]**Although very few of them were tried to be kept as a museum ship in the first half of the 18th century,these efforts could not be successful and none of these ships have reached today.

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Figure1. "Yüzyıllar Boyunca Venedik ve İstanbul Görünümleri", İstanbul Topkapı Sarayı Müzesi ve Venedik Correre Müzesi Kolleksiyonlarından, İtalyan Kültür Merkezi , 1995.

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VIRTUAL REALITY RECONSTRUCTION OF THE ROMAN TOWN CARNUNTUM / AUSTRIA

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ABSTRACT

To demonstrate the potential of archaeological prospection data, computer aided reconstruction is used to illustrate the archaeological interpretation of a part of the Roman civil town Carnuntum east of Vienna. This part is mainly known from data collected during the last three years by an integrated prospection approach combining aerial archaeology, geomagnetics, resistivity mapping and georadar. By combining archaeological knowledge with architectural construction techniques from the Roman period we try to derive virtual reality scenes that can be shown to a wider audience to illustrate a reconstructed scenario of archaeological sites not yet excavated. By using software for architectural modelling as well as desktop virtual reality techniques we create virtual walkthroughs of the Roman civil town of Carnuntum/Austria.

The virtual walkthrough starts from outside the town from the necropolis situated aside the main road leading into town from the south. By passing the town wall with the fortification ditch in front we enter the town still on the main road. We pass several houses (shops, housing areas, working areas), a temple and enter a large public building at the southern end of the forum. After crossing the forum the walk leads through a basilica to a monumental public bath.

The computer-aided reconstruction is a fast and cost effective way to present the archaeological interpretation model derived from prospection data to a wider audience.

1. ARCHAEOLOGICAL PROSPECTION

As archaeological excavation is a very time and cost intensive process, methods that can generate predictive and non-destructive results of possible archaeological sites are of high interest. Furthermore some methods of measurement at ground level can produce highly accurate results that allow a three-dimensional computer based reconstruction of ancient buildings.

The following example of such a reconstruction is given for the Roman civil town Carnuntum, situated near the east of Vienna, Austria.

Figure 1: Roman Civil Town Carnuntum, near Vienna

Since three years, several types of measurements at aerial level as well as on ground level have generated an enormous amount of data that has to be processed with computer based methods to produce visual results that are suited for archaeological interpretation.

Figure 2: Measuring the archaeological site

The measurement itself is done by manually pulling the measuring device across the area of interest along a grid on the ground, which is used as a spatial reference. The values generated by the device are transferred to a mobile computer, which stores the value for further processing.

The several types of measurements (geomagnetics, resistivity mapping, georadar) are combined into gray-level images, where the variation of intensity determines the variation of density of the material below the ground.

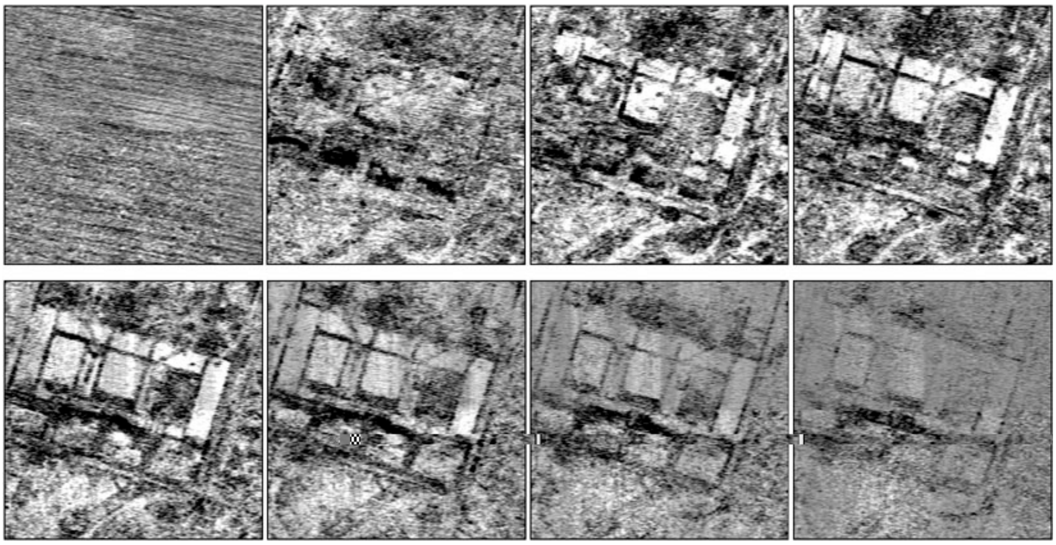

Figure 3: Measurements in various depths of the same area

The images above represent the results of an area at Carnuntum, where each image represents the same area at different depth (about 4m below ground level at the maximum depth). At certain levels the basic structure of a building can be seen very clearly. Now the data is suited for further interpretation. Fur a deeper explanation of the used measuring methods consult [5] and [9].

1.1. Archaeological Interpretation

By archaeological interpretation of the several layers of the building it is possible to derive certain functions of rooms, stairs, columns, canals, etc. This functionality of the building is determined by the shape and material, which can be analyzed from the images. For a detailed description about the relationship between measured values and different types of material read [5] and [9].

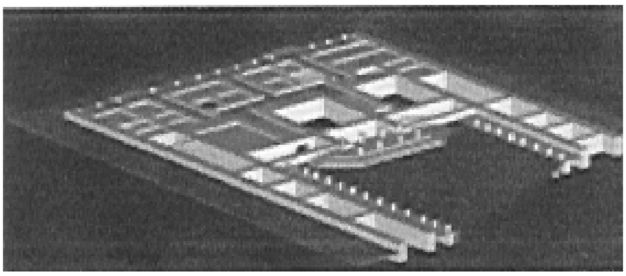

Figure 4: Archaeological interpretation of the measurements

Figure 4 describes the final interpreted version of the measured area. It leads to the basic structure of the basement that is used further for a complete three-dimensional reconstruction, where additional interpretation is necessary to provide plausible results.

2. VIRTUAL RECONSTRUCTION

The continuation of the reconstruction involves further knowledge of roman architecture. Parts of this knowledge is taken by the descriptions of roman architectural principles [8], by the analysis of roman building techniques [3], images of roman excavations and reconstructions of similar buildings [1] and [4].

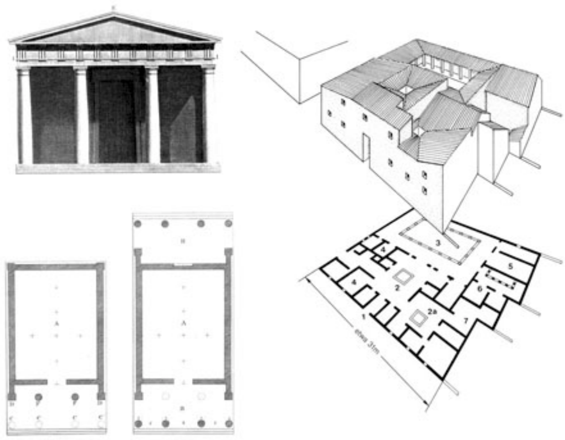

Figure 5: Roman architectural principle [8], Roman building technique [3]

In general the vertical size of the building has been estimated by the measured wall thickness as well as by comparison with known building types that possessed a similar base structure. Also building types could be estimated by taking account plausible roof structures. A porticus was defined were there was measured evidence of column-bases or where it was plausible from the derived building type.

2.1. Managing Complexity of the three-dimensional Model

As the process of reconstruction was enlarged for a part of the town and some library objects, as the roman column [7], were modeled with a high degree of complexity, it was necessary to reduce the amount of data inside the computer to be able to manage the entire project. Since it is not always possible to upgrade the computer hardware with faster processors and a larger amount of memory, which has been done also, other solutions for managing complex three-dimensional models have to be found. The main approach has been to limit geometric complexity by reducing the number of polygons to represent one particular object without great loss of visual appearance. The main idea was to find a reasonable amount of complexity for each object that would allow an accurate rendering from a typical viewpoint, as standing on the center of a street or at the entrance before a temple.

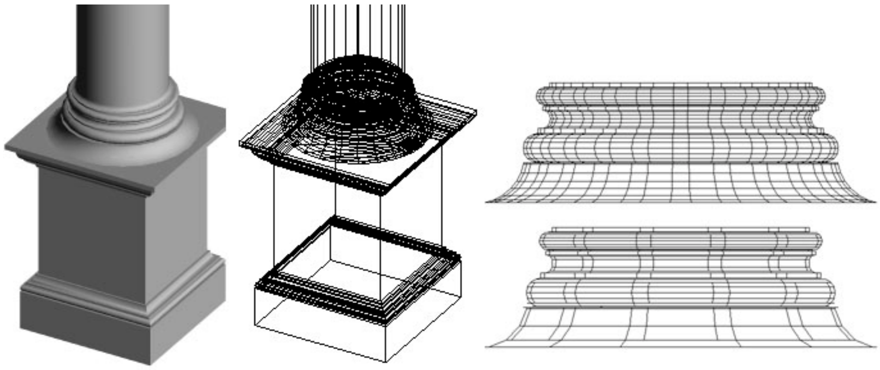

Figure 6: reducing the number of polygons at highly detailed parts

The column shown in the figure above illustrates the reduction of complexity at the example of its curve-shaped base. Hereby an adaptive approximation with polygons with a limited angle between two adjacent polygons has been processed. At the example above a reduction to almost 25% of the original number of polygons could be reached.

2.2. Adding Textures to enhance Visual Appearance

To achieve a high degree of detail, the application of textures on surfaces is a very successful approach for enhancing the quality of computer generated images. In this work images from excavated parts of Carnuntum as well as from buildings in Pompeji [6] have been used. After scanning the photographic images, textures have been generated by removing perspective distortion and by equalizing the influence of light on the surface. These textures were then used repetitive with proper alignment and scaling on the appropriate polygons.

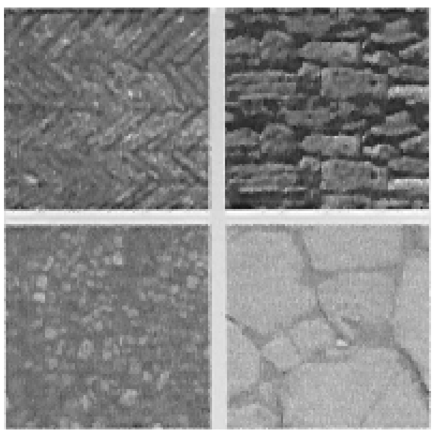

Figure 7: Example of textures used for floors, streets and walls

3. RESULTS

The current state of the three-dimensional reconstruction from the newly discovered parts of Carnuntum demonstrates some interesting properties about this multidisciplinary approach. The combination of geophysical prospection methods, image processing, archaeological knowledge, architectural principles and computer graphics can generate virtual images of non-visible archaeological objects that might be interesting for further investigation. It is a very time and cost efficient approach for archaeological research as well as a non-destructive method for the existing environment above archaeological sites. Furthermore the resulting images are also very well suited for public presentation and for research discussions for experts.

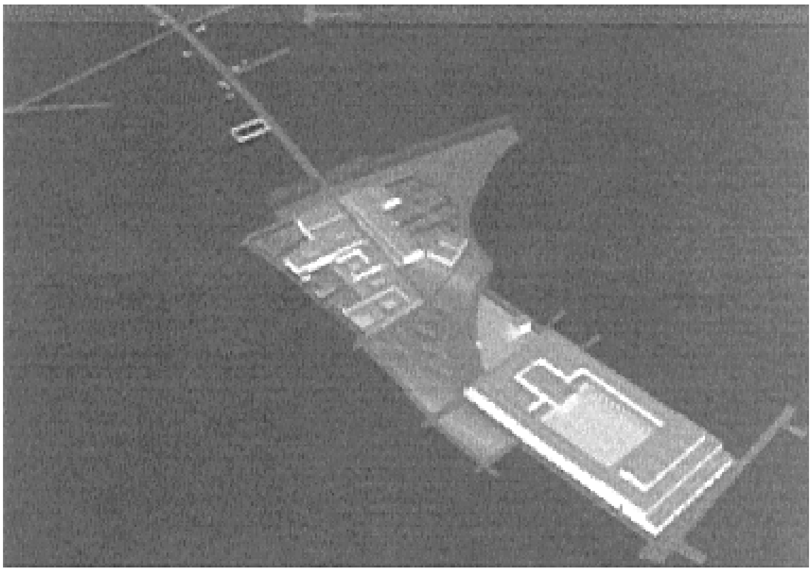

Figure 8: Overview of the reconstructed area of Carnuntum

The image shown in figure 8 illustrates an overview over the currently investigated area of Carnuntum. It shows a long road that leads through several burial sites to the entrance of the town protected by a stone wall, then it continues through a street of buildings for living and working to a square or open place in front of a large public building. At the side entrance of this public building a temple has been found.

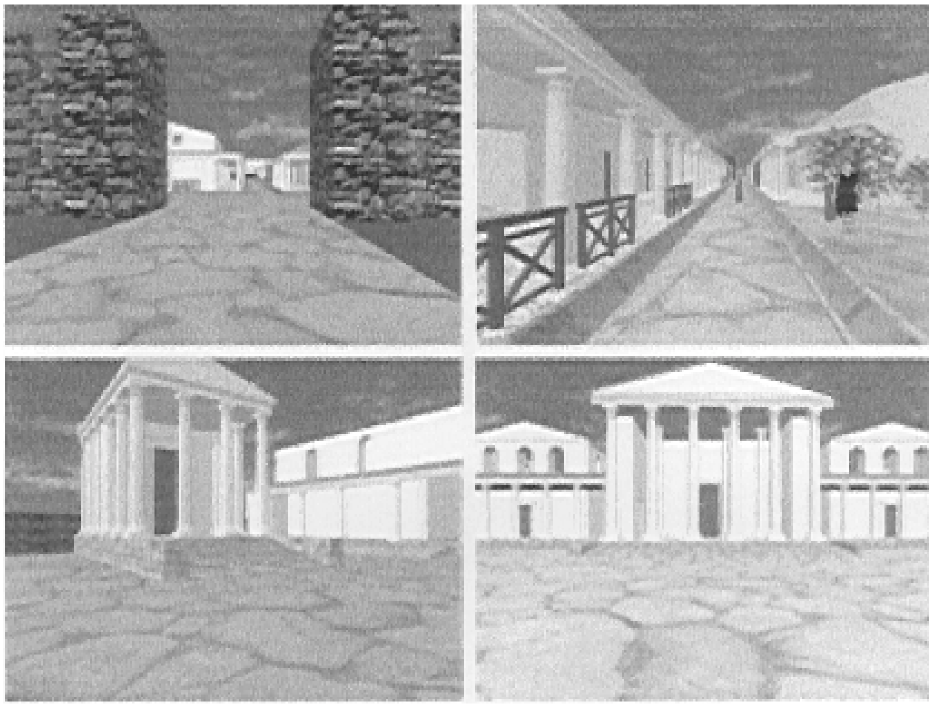

Figure 9: Examples from the reconstructed part of Carnuntum

Beside the images above, which have been rendered from a street perspective, a virtual walkthrough has been generated using the QuickTime VR technology from Apple Computer. Hereby it is possible to make an interactive trip through this virtual part of Carnuntum. Future extensions of this project might be to build a complete virtual model of the town Carnuntum, including already excavated buildings, as well as newly discovered objects in this area.

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A RESTITUTION PROPOSAL FOR DONUKTAŞ - TARSUS

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ABSTRACT

Donuktaş is one of the important protected historic structures in Tarsus. It has been dated to the second half of the second century on the basis of architectural finds. Today the structure is gigantic. It measures 42.90x116.30 m. This impressive monument is in a ruined state today. It is accessible only through a section that forms the substructure of the original building.

The data obtained from the archaeological excavations lead by Prof. Dr. Nezahat Baydur* between 1982-1992, makes it possible to formulate a proposal for this ruined monument. Through examination of the data uncovered by excavation and through comparison with buildings of the same period, it is possible to conclude that Donuktaş was a peripteral temple with 10x21 columns. In this paper, the proposed restitution is based on the archaeological data evaluated analogically.

1. INTRODUCTION

Donuktaş is located outside the ancient city of Tarsus, on the south bank of the Cydnos (Tarsus) Brook. Tarsus has grown outside its ancient walls and today the location is known as the Tekke quarter [3], 20 meters above sea level (Figure 1). Donuktaş has been dated to the second half of the second century on the basis of the architectural fragments found at the site and its decorative features [3].

Archaeological digs at Donuktaş started in 1982 under the leadership of Prof. Dr. Nezahat Baydur and continued until 1992. The digs were carried out under difficult conditions. Unfortunately, no written document or inscription identifying the building was obtained during the excavations. Moreover, it was clear that the structure had been destroyed at a very early date, before being fully completed.

Figure 1: Aerial view of Donuktaş

2. ARCHITECTURAL DESCRIPTION

According to excavation data, the building stands on a rectangular podium measuring 60.50x145.00 m and is oriented northwest-southeast. The building consists of eight blocks; four of them are independent while the remaining four are attached to one another at the same or different levels. As seen in Figure 2, the blocks named F, G and H are continuous and have the same height. C is attached to the blocks F and H at lower levels. The blocks are approximately 6.50 to 7.00 meter high from the ground. Only block A with its sloping and flat surfaces differs from the others as a stair block.

Block A, with 22.50 m width, 41.20 m length and 11.56 m height have three landings of 1.30 m and two flights of stairs of 6.00 m in length. There is a passage between blocks A and B with a width changing from ~7.50 m at the ground to ~5.00 m at the top. On block A there is a circular votive altar with a diameter of 6.60 m and a grave dating to the late Roman period.

Block B, has a 6.00-6.20 m width, a 39.70-39.90 m length and a ~10.95 m height. The block B is unconnected to the others. On the northwest façade of block B, stones as if springing of a vault were found in their original position. The southeast façade of block B descends to ground level vertically. The passage between blocks B and C is 3.20 m wide.

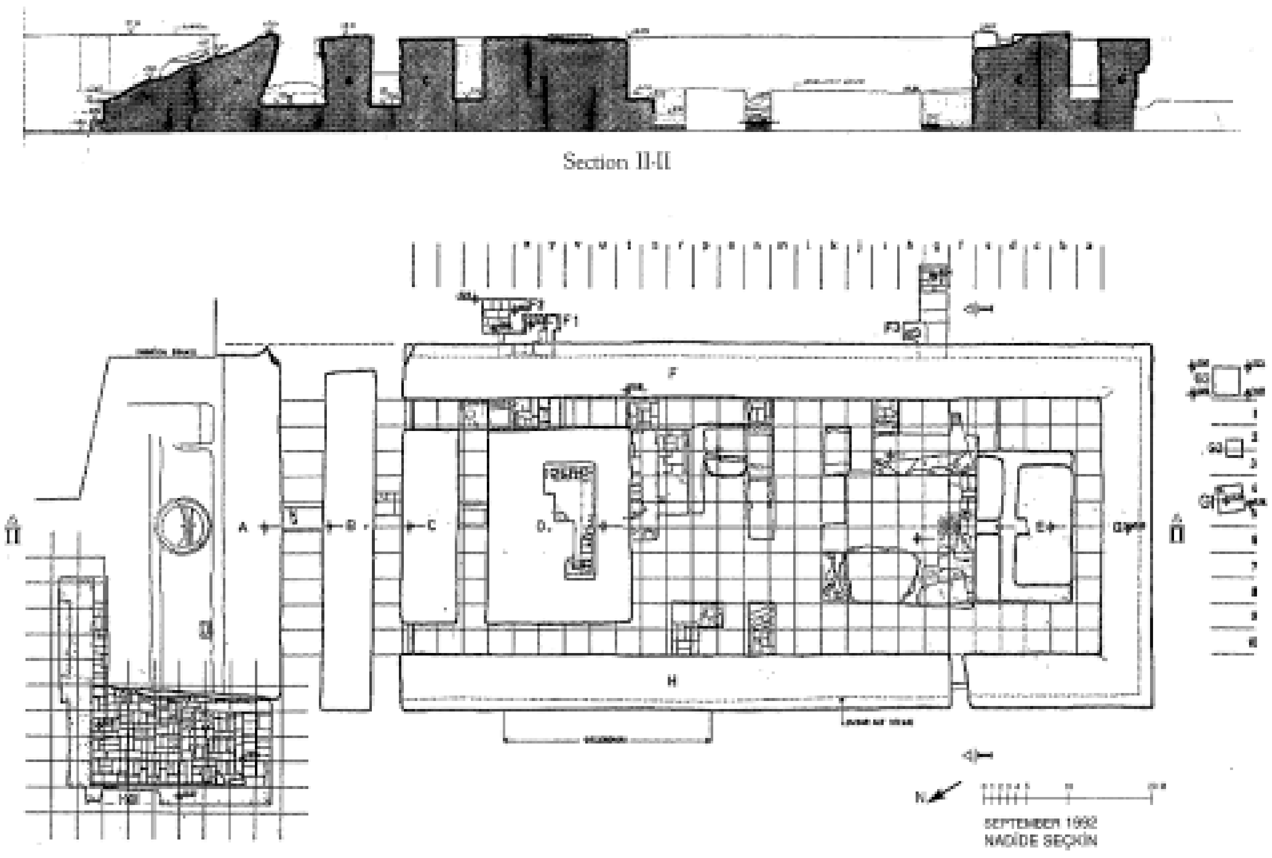

Figure 2: The plan and section of Donuktaş, 1999

Cella, has a 29.90 m width and a 75.60 m length. The cella floor of limestone blocks placed over an approximately 40 cm thick layer of cement is at -10.00 m level when the top of block G is taking as ±0.00. On this floor, there are vault like fragments which don’t have any relationship with the other blocks of cella.

Block C, has a 6.90 m width, a 23.00 m length and a ~8.15 m inner height. The block C forms the northeast façade of the cella, and it is attached to blocks F and H at low level. Between blocks C and D there is a passage which is 3.25 m in wide.

Block D, has a 17.10-17.50 m width, a 23.00 m length, and a ~7.00 m inner height. The top of the blocks C and D are reached through an earth-fill in the form of a ramp found between the blocks C, D and H.

Block E, has a 11.40-11.70 m width, a 17.75 m length and a ~7.00 m inner height. The block E seems to be the highest of the building blocks. There are asymmetrical partitioning of different heights above it. Its side distance to the blocks F and H is 6.00 m, and rear distance to the block G is 3.60 m.

Block F, has a 6.50 m width, a 88.45 m length, a ~5.00 m inner height and a ~7.60 m outer height. The building block F forms the southeast wall of the cella. It is attached to the block G at the same level. The external façade of the block F has an inward vertical steps. Therefore the 6.50 m top width becomes 5.00 m on the ground level. The cella façade of the block F descends to ground level vertically.

Block G, has a 6.30 m width, a 42.90 m length, a ~5.40 m inner height and a ~7.40 m outer height. The building block G borders the cella in the direction of the southeast. The external façade of the block G has an inward vertical steps. Therefore the 6.30 m top width becomes 5.30 m on the ground level. The cella façade of the block G descends to ground level vertically.

Block H, has a 6.50 m width, a 88.80 m length, a ~6.00 m inner height, and a ~7.15 m outer height. The building block H forms the northwest wall of the cella. The external façade of the block H has an inward vertical steps. Therefore the 6.50 m top width becomes 5.50 m on the ground level. The cella façade of the block H descends to ground level vertically.

3. CONSTRUCTION TECHNIQUE

The building blocks rise over the stone blocks laid on a ~40 cm thick concrete floor. The building blocks, which still exist today, were made of Roman concrete in which pieces of stone and conglomerate of various sizes were used as aggregate and cast in place in the form of strata. As this method was used widely, there is the cast concrete section that serves as the core between the walls. The interior and exterior surfaces of walls are made of plain hewn stones without mortar [1, p.42]. At Donuktaş, the inner face of the stone wall which does not exist today was stepped inward to the core, ensure that the concrete section adhered to the stone face. By taking up the stones, the external surfaces of the concrete section appeared stepped (Figure 3) [1, p.45]. Unfortunately during the excavation no evidence related to the existence of a wall on the inner face was found.

Figure 3: The stepped external surface of Block G

From outside, blocks F, G and H seem to be constructed together. From inside, the vertical intersection lines at the corners of the cella are very clear. The top of the wall shows no traces of being cracked. When the blocks are observed, it seems that the casting is made at one time in both the outer corners and on the upper surfaces. In large blocks, the layers of casting are at different levels. The casting was done in rows that are generally 60 cm high.

In block A, there are 11 cast levels that are repeated at specific intervals. The external section of block A and the destroyed staircase section were completed with block stones fitting the height of the cast levels. The stones were used without attaching by clamps or mortar.

The dimensions of the aggregate in the cast cement creating the inner structure of block A shows variations (Figure 4).

Gravel	*(cm)*	*4x6, 5x6, 5x7, 6x6*
Crushed stone	*(cm)*	*18x11, 11x15, 8x18, 12x17*
Conglomerate	*(cm)*	*12x16, 15x17, 12x13, 14x14, 14x15, 14x15, 8x12, 11x18*

Figure 4: Block G, texture of cast concrete

4. RESTITUTION PROPOSAL

The data obtained from excavations made outside the building, in the krepidoma and in the cella, make it possible to formulate a proposal for the restitution of Donuktaş**. Excavations revealed the measurements for the width of the podium and the krepidoma. In evaluating the extant data in this connection, one could say that Donuktaş could have been a peripteral temple with 10x21 columns, set on a podium with dimensions of 50.70x106.80 m [3]. As seen in Figure 5, in evaluating the measurements for the columns on the front façade that have to be on the centerline of the blocks F and H and on the same level with the structural walls, the column interval comes out to be 42.90-2(3.25)=36.40 m; 36.40 m / 7=5.20 m.

Starting from this measurement, it is necessary to add an area 3.90 m wide to the exterior surface of the existing blocks when placing 10 columns at the same interval on the northeastern façade. So the width of the podium obtained is 3.90+42.90+3.90=50.70 m. In block A the first step of the krepidoma extends 8.80 m further out than the external surface of the blocks. In the area remaining related to these measurements, the width of the krepidoma must be 8.80-3.90=4.90 m. The width of the steps are 27.22 cm if there are 19 steps with a height of 61 cm in order to ascend to the ±0.00 m level, the block above, from the -11.56 m level that is the plastered cement floor in the krepidoma. One should not forget that these calculations were made according to measurements taken from the extant uneven substructure.

Using a similar approach, the measurement from the middle of the block B to the axis of the section that has been added to the block G is: 3.00+3.30+88.80+1.95=97.05 m. When this distance is divided into 20 column intervals, the measurement for the column intervals on the long side is 97.05 m / 20=4.85 m.

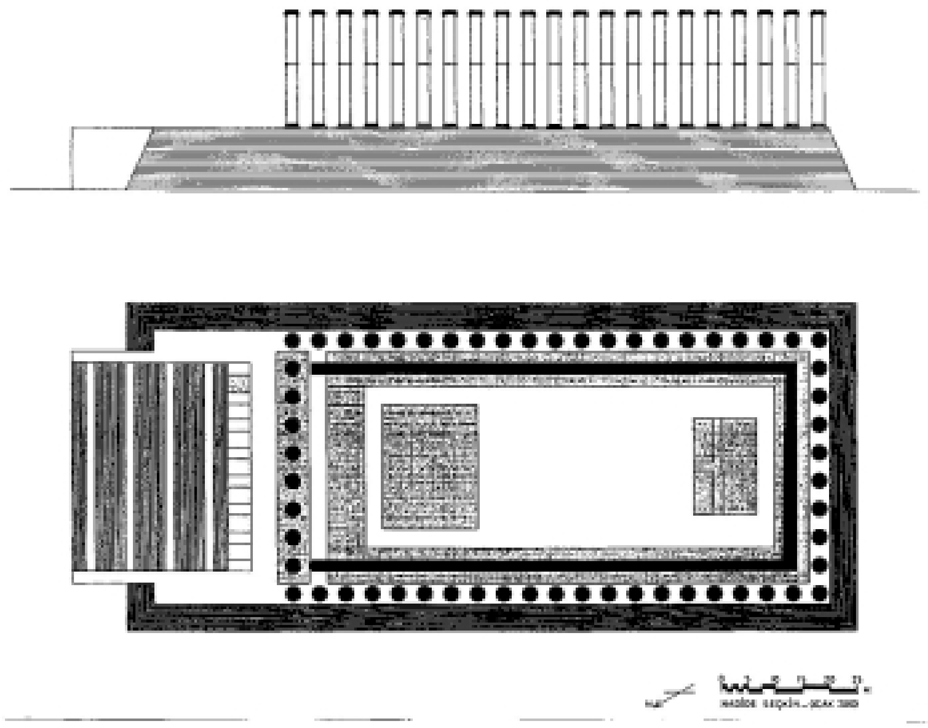

Figure 5: Donuktaş, restitution proposal plan and side elevation

Using the existing slopes of block A, the block length becomes 33.00 m. When the measurements of the existing flight of stairs and the landing are taken into consideration, the length of the missing staircase section is 10.30 m. One can consider that within this measurement the design calls for a 3.00 m stair + 1.30 m landing + 6.00 m stair, suiting the existing measurements. So at the head and foot of the staircase, there are two sets of stairs at 3.00 m, three sets of stairs at 6.00 m between the two sets and four landings of 1.30 m length. On the first and last stairs that are 3.00 m in length, there are eight steps and as for the 6.00 meter-long stairs in between -- assuming there are 16 steps -- there is a total of 64 steps. As for the height of each step it can be calculated at 11.56:64=18.06 cm. The width of each step will be 600:16=37.5 cm because the final step is that of the landing and the length of the landing will be 130+37.5=167.5 cm. There are pedestals that are partially preserved on top of block A, on the same axis of columns on block B (Figure 5).

It is difficult to compare Donuktaş with buildings of the same period [2, p.42], [4, p.337], [7], [8, p.48 and Ch.III pl.V]. Unfortunately there is no surviving example in which it is possible to see the relationship between the substructure and the upper structure. The closest example is the Temple of Didyma. With the measures and proportions that have been accepted in the proposal for the restitution plan for Donuktaş, similarities exist between it and the measurements and proportions of the Didyma Temple. These measurements and proportions are seen in the Table 1 given below.

Table 1: Comparison of measurements and proportions used at Donuktaş and the Didyma Temple

Measures / Proportions	Donuktaş	Didyma Temple
Podium dimensions (m)	50.70x106.80	51.13x109.34
Krepidoma width (m)	4.90	4.50
Column diameter (m)	2.10	2.022
Column heights (m)	19.95	19.70
Column intervals (m)	5.20 (short side) 4.85 (long side)	5.301 5.296
Step width (cm)	37.5	37.5
Podium proportions	50.70:106.80=**0.48** 106:80:50.70=**2.11**	51.13:109.34=**0.47** 10.9.34:51.13=**2.14**
Column interval proportions	4.85:5.20=**0.93** 5.20:4.85=**1.07**	5.296:5.301=**0.999** 5.301:5.296=**1.001**
Column interval / Column diameter	4.85:2.10=**2.31** 5.20:2.10=**2.48**	5.296:2.022=**2.62** 5.301:2.022=**2.62**
Column diameter / Column interval	2.10:4.85=**0.43** 2.10:5.20=**0.40**	2.022:5.296=**0.38** 2.022:5.301=**0.38**

Information about concerning the columns used at Donuktaş is very limited. A drawing on a 1/1 scale was made from a piece of broken marble belonging to a fluted column shaft found on block A (Figure 6). The plan of the complete body is given in Figure 7. As seen in the figure there are 23 flutes in the body of the column that is d=1.05 m in radius***. In the proposed restitution, the column radius has been accepted as 1.05 m. The column diameter / column interval is 0.43 and 0.40. When this proportion became 0.44 according to Vitruvius -- including the column base and capital -- the column height is h = 9.5 R [5, p.19 ff], [6, p58, 60]. According to this, the height of the column has to be 19.95 m, including the base and capital [4, p.337].

With the assistance of a piece of large dimensions found on block A (a torus and a half trochilos) (Figure 8) and a portion of a torus found in bench No. XII, a proposal for the cross section of the column base is presented in Figure 9****. When the aforementioned two architectural fragments' diameters are considered, the column base used at Donuktaş resembles the column bases at the Antonius

Pius Temple at Sagalassos [1, P.97], the temple at Komana [1, P.217] and the first temple at Kaunos [1, P.177]. In preparing a proposal, one has to look carefully at the proportions of the column bases belonging to the aforementioned sanctuaries, and thus the height of the column base at Donuktaş has to be 79 cm. There is no evidence related to a pedestal at Donuktaş.

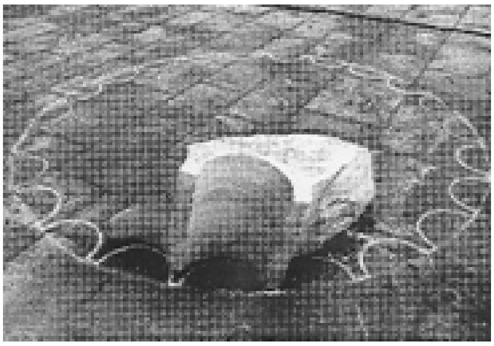

Figure 6: The marble piece of column shaft

Figure 7: Donuktaş, plan of column shaft

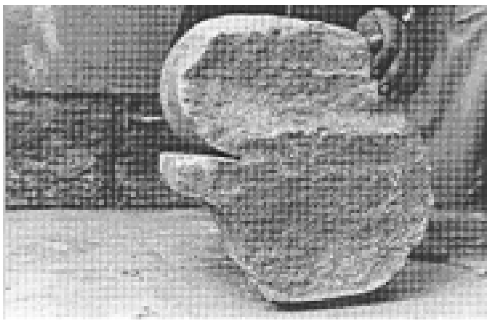

Figure 8: Marble piece of column base

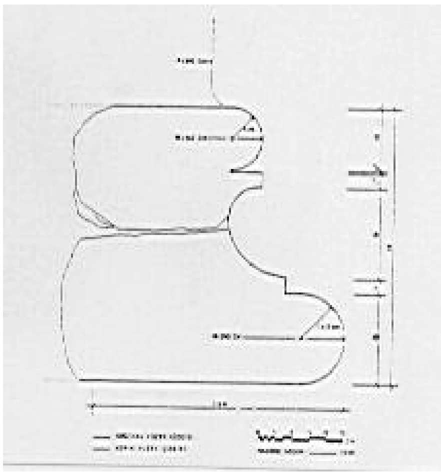

Figure 9: Restitution proposal for column base

During the excavations at Donuktaş, no revetment was discovered in-situ.

Unfortunately, information about the form of the roof at Donuktaş and the roofing material is very limited. During the excavations most of the marble and fired earthenware tiles which must have been used to cover the roof were found in pieces [1, p.52].

Over a ten-year period, 10% of the building area was excavated. Documents illustrating the original building at Donuktaş either in writing or visually are unfortunately very scanty. This paper has tried to interpret and evaluate the data obtained by archaeological excavation. Architectural data provided by drawing analogies from similar temples; has made it possible to develop a restitution proposal for Donuktaş.

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* Retired professor from Istanbul University, Department of Classical Archaeology. I am indebted to her for her permission to make this presentation.

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*** For this drawing I am indebted to Mary E. Strain, the Icomos US trainee for 1999.

**** I have to thank Yahya Karslıgil for the calculations.

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EFLATUNPINAR: FROM LOAD-BEARING TO POST-LINTEL STRUCTURES, EMERGENCE OF STATICS

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ABSTRACT

The unique presence of the Hittite Kingdom water shrine structure at Eflatunpınar, its construction dating back to the Late Bronze Age (L.B.A.) of 13th century BC, lying 5 km to Beyşehir, Konya, has been known for more than 120 years now. Research aiming to decipher its meaning, use and importance go hand in hand with recent explorations about the original state and structure. The relation to the incomplete, unerected 7.40 m tall Fasıllar monument 50 km away from the site; cosmogonies related to the geometric plan organisation and figures; the construction technology, craftsmanship and their semantics; the symbolic value of the emblems and signs on the carved and constructed surfaces; are all fields full of promise.

As other Early and Late Hittite Kingdom structures and architectural works, the sanctuary is a piece of worship to the concrete substance of the rock itself in the name of an earthbound religion. To worship the mount, to worship the sacred representations of an earthly power, to worship the deified King through the representation of rock-cut substitute statues, are all but part of the consequent linear understanding. Contemporary with the Old Kingdom at Troy; preceding the 'culture' of the ancient Greek civilization much appreciated by Orientalist scholars, Hittite architectonic work promises new horizons: The 14 partite stone façade of the upper monument presents forth deities sheltered by lintels supported by demon figured columns, 9 centuries before the caryatid figures of the Erechteion at Parthenon; while the lower monument offers links to Yazılıkaya, Hattusha.

1. INTRODUCTION

The interest in this Hittite water-shrine monument started to gain popularity with the account given by Perrot in 1890 [43] but he mentions that Hamilton had described the monument after an excursion in Anatolia, in 1885. We know that Sarre [45] also briefly described the monument and its nearabouts in 1895. After the proclamation of the Turkish Republic and in line with the extensive endeavour to delineate the limits of an objective history of Anatolia, excavations and research were initated and encouraged by the State, both via national and foreign archaeologists, this time for national concerns as well. Güterbock, Bittel and Naumann wrote about the monument during 1940s and 1950s [18]; however in 1962, Mellaart [37] published an interpretation about the possible uses and the importance of the building, trying to relate it with the nearby statue at Fasıllar village.[19] The interest in Eflatunpınar seemed to fade out, when after 1995, the regional authorities at Beyşehir decided to recover what remained there. The presented study is an outcome of the author's growing personal indulgence on the meaning, interpretation and decipher of the monument, but as an architect, supported by annual visits starting 1998. Meanwhile surface research and recovery which was started in 1998, also supports the estimations and 'reading' of the author, about the physical properties. The monument is not only revealing in terms of Hittite routines, signification of religious beliefs, which may lead to a re-writing project in history of architecture, but structural analysis of the components may reveal about the relation of meaning in architecture to function of parts.

2. PHYSICAL DESCRIPTION

Four or five km to the Beyşehir (Caralis) lake, the area is still used by villagers to water their animal, and serves as recreation and picnicking spot for the citizens of Beyşehir.

Figure 1

2.1 The Site: Location and Architectural Elements

By asphalt road, the site is found upon a descent; where to the west there is a mound, which was defined both by Mellaart [37] and Naumann [40] to be a tumulus showing the ancient settlement. It is true that, still in 2000, the ground surface is covered with fragments of pottery. The hill to the south has the new village settlement. The spring water feeding the pool, is also feeding the small streamlet flowing westward, among the hills to the fields and finally to the Lake Beyşehir. (**Fig. 1**)

The trachyte stone façade, which faces the south at the edge of the sacred pool, is not remains of a building, but the complete original monument. (**Fig. 2**)

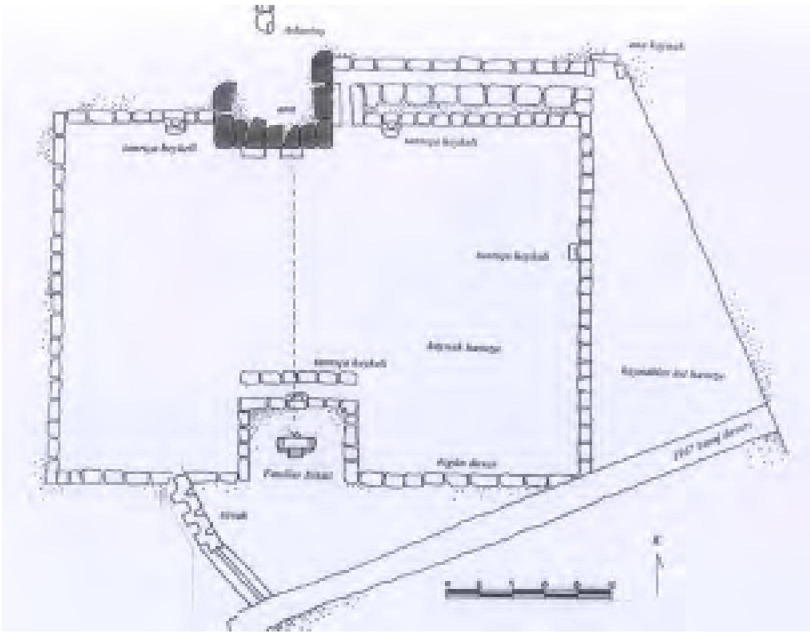

Figure 2. Site Plan Reconstruction Proposal, A. Cengizkan [19]

The side walls of the pool are enveloping this façade, having a goddess statue on both sides and are not so high as was suggested by Naumann. [40] In 1967, the pool was transformed into a small irrigation dam, which increased the water level and thus, the potential resistance of the under-water stone parts against weathering. Contrary to some authors, it is now clear that the monument, the pool and the sluice all belong to the same date. Across the stone monument, there is a small elevated terrace-like land set-forth into the pool, with another stone goddess with with aureole, located out of symmetry with the central axis of the monument.

The sluice, a monolithic piece of stone, (**Fig. 3 and 4**) which seems to be removed from its original place, is located now at west end of the dam, and we have discovered that it has four bull-heads [18, 19], one of them being broken but still on the site. The detailing of this sluice may be promising to understand about

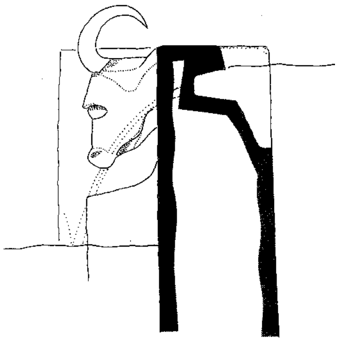

Figure 3. Section Sluice Figure 4. Reconstruction Perspective Sluice [19]

how and under which circumstances this shrine was used.

The orthogonal pool must have been not so deep, and it's still fed by a water source, which makes one think it may not have lost its original medium sized capacity.

2.2. Stone Masonry Monument

During the before mentioned century of interest, research made on the physical characteristics of the monument were limited to the appearance of it. (**Fig. 5**) Sarre refers to a "14 partite monument, the façade having a measurement of 7.02m by 3.30m." The photograph shows us that the water level of the pool in front of the stone masonry façade is high. Perrot, gives a more accurate account, "6.83m by 3.85m" depending on Ramsay, and Naumann still as "7.00m by 3.96m"; and we measured it by 1998 as "7.04m by 4.89m". [9] It is quite obvious that the drainage achieved during research in summer excavations yields a higher façade, at least 1 m taller.

Figure 5. Façade of Eflatunpınar in 1890, G. Perrot, [43]

It expresses mainly two figures, sheltered by a "pair of winged sun-discs, represent(ing) a god on the left and a goddess on the right, both seated".[37] The male figure's arm is raised up, though both are quite weathered, and the female has an aureole, which is fairly visible. According to Akurgal [4, 5, 6], the male figure being depicted on the right hand side, reflects the manner of respecting female deities and goddesses in the very early Anatolian cultures, which is still living in recent Anatolian culture, even during official ceremonies.

On the top there is another monolithic stone, depicted as a winged sun-disc, part of the disc missing due to weathering or other environmental reasons. The 5 couple of column-like stones, resting on top of each other, with standing figures of deities sculptured on the main façade, both arms raised up, with gowns and shoes still visible.

On the lowest layer of visible part of the monument, there are 5 blocks of stones again, but the ones on both side ends elongated now and having the depiction of two heads of gods, instead of the central having one god, and the stones under the god and the goddess, acting like pedestals, protruding forth with the god and the goddess. The bust-like depictions reflect more facial features than of the body, where for all the five cases, figures are carved bringing their hands and arms together as to form a shelf in the front.

It is known that the lower-most part of the monument which was under water level until, it seems, 40 years ago, was also excavated during the surface research started in 1998 as summer research. What was found is not surprising: another line of stone masonry, depicted with their caps like the mountain god Tarhuntas at Yazılıkaya on the façade again.

2.3. Construction Technique

As with other Hittite masonry buildings, it is thought that Eflatunpınar was also built with stone transported from neighbouring quarries. The probability that the

Figure 6. Detail of Blocks Prepared for Easy Handling

blocks were displaced with coarsely carved surfaces, and articulations were made on the façade in-situ after the erection, is also supported by the almost hand-carved signs to the back of the structure, which should have facilitated the easy transport and installment of stones. (**Fig. 6**) Similar traces may easily be traced at Hattusha, Fasıllar, Gavur Kalesi and Alacahöyük. Familiar with the wood-cut techniques and tenon-like ending wooden statutes and sculptures of gods and goddesses, the Hittites should have easily transferred this technique to stone-masonry building.

3. TECTONICS AND SEMANTICS

Architectural configuration of the 14-partite stone-masonry monument can now be handled in terms of the succession of stones, their load-bearing functions and the correlation of the depiction on the façade and the stone's location.

It can be stated that, there are two frames governing the construction of the monument: The outer frame (depicted in *red* shade; **Fig. 7**) is supported by the wide mono-block foundation stones at both ends of the structure (as supports), as if to offer wider-and-larger widths and depths for side loads (created by) due to earthquakes. At the second layer of stones, the tree-like branching of the vertical load-bearing element is achieved; the inner parts are creating the vertical load-bearing elements of the inner frame; whereas the outer frame is constituted by the two blocks. The horizontal 'span' is covered by another but this time long (7.50 cm) mono-block stone, with the depiction of a "winged sun-disc".

On the other hand, the inner frame may be deciphered to have two (or three, with the central one) supports (depicted in *blue* shade; **Fig 8**). The vertical load-bearing elements are two blocks again, but the horizontal 'span' this time, has "a pair of winged sun-discs" on the façade, each settled on the head of the god-goddess figure depiction. However, organisation of the frame gives the impression that, while the importance of the god-goddess figures is emphasized, the god with the right arm upraised and the goddess with an aureole, the structure is working to decrease the weight of the load on both, coming from the real dead-load of the stone blocks.

Figure 7. Outer Frame

Figure 8. Inner Frame

Such a 'reading', or 'decipher' should not be taken to be coincidential:

If we focus on the images depicted on the façade, once more, we realize that the 'load-bearing vertical elements' are offering us images of deities, similar to the Yazılıkaya mountain gods in procession, but each figure as "both arms raised up, with specific gowns and shoes"; organized as if to give the impression that "they make the loads bearable" for the sake of the god and the goddess. Let's call these 'vertical load-bearing elements' as "columns"!

On the other hand, the 'horizontal load-bearing elements' are defined as "single, and/or, a pair of winged sun-discs", where divinity as connotations of the wings and the sky and being located to the upper-most part of the monument are supporting the meaning of the structure, with the flow-of-time is represented by the sun-disc; while still, their mono-block construction is related to the continuity of the 'horizontal load-bearing element', which we may now easily call as the "beam"!

Coming back to the lower part: The figures as divine busts, are depicted as "hands and arms brought together", thus managing the "easy" construction of shelf like protrusions, on which possibly offerings were put by worshippers; in other words, they might have acted as "libation" shelfs or cups. [7; 51] These busts are described with faces at repose; as if they are indifferent of the load they do carry but with the awareness of their task, and support!

Still, there is the lower-most part of the façade, which, though revealed, we were unable to document by photographs.[18; 19] However, we know that the description of the images are like the ones at Yazılıkaya, depicted as mountain symbols under the feet of Tudhaliya IV, the last king of the empire. (**Fig. 9**) However here, they are the caps of small mountain deities, in congruence with the understanding of worshipping the mountain; once more "the substitute and the symbol being one and the same thing": the divine, is the real support, or foundation, which is the Earth, thus, the World! [20]

Figure 9. Tudhaliya IV

Figure 10. Tudhaliya IV; *'calmush'* [19]

All these are beyond coincidence. Further, the similarity of the signature of the king (called the *calmush*), its unique but unbalanced symmetrical and facial formation to the facial formation and symmetry of the monumental façade draws one's attention! (**Fig. 10**) The signatures are always sheltered by the "winged sun-discs" at the top, double sun-discs representing the king claiming his own divinity in life-time, which is valid for Tudhaliya IV only.

We are defending that the concrete procedures in the making of this monument have paved the way for a clear awareness about load-bearing capacity of structures. Akurgal was one of the first researchers to draw attention to Hittite art and architecture to precede long-and-far appreciated Greek art and architecture. [1; 2; 3; 5; 6] His emphasis on the load-bearing property of the Late Hittite Kingdom column excavated at Tel Halaf (**Fig. 11**) and its affinity with the Caryatid figures (**Fig. 12**) at Erechteion (447-431 BC) near the Parthenon, at the Acropolis, Athens; should now be once more under focus. [5] We are defending that, the Eflatunpınar monument, as a masterpiece of the Hittite Kingdom, which was constructed 13th century BC in Anatolia, fore-visioned the load-bearing vertical elements represented as human figures of the 5th century BC in Greece! [29; 30; 31; 38; 39] More than anything, the attempt seems to clarify the understanding of *techne*, leading to the foundation of 'statics in architecture'.

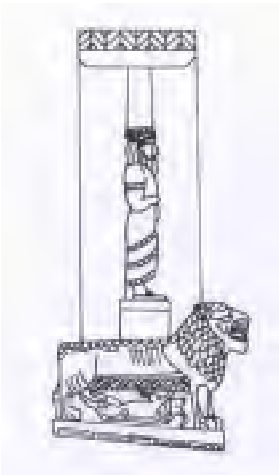

Figure 11. Tel Halaf

Figure 12. Erectheion; November 2000

4. CONCLUSIONS

The significance of the L.B.A. Hittite monument at Eflatunpınar is multi-faceted. Its relation to nearby monuments and sites to enlighten a certain era of Hittite history and architecture; the promising points inherent in its own features to decipher cosmognomic characteristics of Hittite living; the construction steps and traces which reveal not only about ancient building techniques but also leading to interpretations to develop hypothesis as to correlate the development in artistic achievements and *status-quo* of *techne*... It is thought that learning from Eflatunpınar will be a continuous process in the coming years.

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CHAPTER II

DOCUMENTATION
of
ANCIENT STRUCTURES
and
HISTORICAL ENVIRONMENT

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GIS BASED DOCUMENTATION SYSTEM FOR CULTURAL HERITAGE SITES

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ABSTRACT

Paper analyses relation of mass tourism to cultural heritage stability based on knowledge taken from Central European experience. It presents a sample of the contemporary situation in the Czech Republic with the relevant economy consequences. There are studied impacts and risks generated by a high attendance of visitors to cultural monuments as well as risks to visitors and their reduction possibilities. The contribution also deals with the role of cultural tourism in social and economic stability of cultural heritage sites and the needs of tourism in development planning. It is concluded with some recommendations for creation of tourism management policy.

1. INTRODUCTION

The documentation system for small historic cities and villages has been developed as a practical tool for:

- preparation of their strategic plans,
- preparation of land use plans including safeguarding and enhancement plans,
- management of their sustainable development,
- documentation of their historic values and management of monuments,
- inventory of cultural heritage in a territory,
- preparation of financial and regeneration plans for built heritage,
- heritage impact analysis of investment in a territory,
- elaboration of information issues for citizens and visitors,
- possible connection to public information systems (Internet) research purposes.

The system has been tested in one of the World Heritage Cities – Telč in Moravia under support of the Czech Ministry of Culture and the Council of Europe.

2. GENERAL FEATURES OF THE DOCUMENTATION SYSTEM

2.1. Basic approach

The system has been designed as a set of special databases attached to a true geographic information system (GIS). For small cities and villages, a cadastre (land-register) digital map is used as a reference base, which enables to attach relevant data – alfa-numeric, textual or graphical-- to individual lots or objects projected on the map, [1].

It is supposed that the vectorisation of cadastre maps all over the country will be terminated by the year 2005 and then the digital maps will be available for all cities and villages in the Czech Republic.

2.2. Links to international standards

The system conserves namely the completeness of recorded data given by the recommendation of the Committee of Ministers of the Council of Europe No. R(95)3 on co-ordination of documentation methods and systems for historic buildings and monuments of architectural heritage (1995). Similarly, international standards for archaeological monuments and sites, as well as mobile objects are applied and respected, [2].

2.3. Links to national standards

National standards and recommendations of territorial identification and cultural heritage inventory or documentation are especially respected. In order to facilitate the transfer of data already recorded into the developed documentation system, a full compatibility with ministerial or regional databases for protected monuments has been maintained.

3. CONTENTS AND DESCRIPTION OF DATABASES

3.1. Data structure

Data for the documentation system of historic cities are collected in a broad spectrum, which enables to construct targeted databases and outputs. Those databases are grouped into thematic clusters for the sake of brevity.

There are recorded all available pieces of information on monuments, objects of historic or cultural interest and their context links in a territory of interest. An important part of the system is devoted to economy data which can be utilised for planning, as well as for heritage impact analyses or potential evaluation in historic cities. Their interaction with environmental issues influencing conservation or maintenance costs is taken into account, too. The system provides users with references to documents kept in different archives.

3.2. Thematic groups of data

The main eight groups of collected data and a rough description of their contents are presented further. A detailed structure of actual records is dependent on the type of a unit under consideration and it is modified automatically at the moment of its insertion.

3.2.1. Unit definition

This block contains basic identification data, related units references and identification of the data provider. The types of units selected are elements, objects, functional ensembles, historic ensembles, public spaces, and territories.

Element is defined as a part of an object or a parcel of land. It might be a part of a building, spatially defined (wing, tower, gate, etc.), a structure (vault, pavement, door, etc.), an artwork permanently built into the object (fresco, stained-glass window, etc.), or a vegetation or landscape arrangement of the lot (balk, terrace, pavement, etc.).

Object is considered to be a spatially enclosed or functionally independent building (house, barn, fencing wall, etc.), a parcel of land, a spatially independent work of artistic (statue, etc.), cult (cross, Crucifixion column, tomb, etc.), technical or other functional quality (milestone, well, etc.).

Functional ensemble groups objects and lots spatially or technically joined and intentionally linked to fulfil a common function (house with auxiliary buildings, fences, and garden; farm; water mill; sculptural complex; fountain with connected water work; etc.). Functional ensemble is typically identified by one land-register number.

Historic ensemble contains groupings of generally independent objects and lots, each having an individual function, joined together by historical, cultural or geographic relations (road of the Crucifixion with the Calvary in a landscape, historic core of a settlement, subway network, housing estate, etc.).

Public spaces include squares, streets, town parks, embankments, lakes and rivers, etc.

Territories denote parts of the Earth surface defined by laws, approved in urban planning documents or other codes and composed of land elements (Town Heritage Reserves, Town Heritage Zones, Landscape Heritage Zones, urban zones, etc.).

This block also contains so called identification number which uniquely determines the recorded entity. It is composed from the locality relevant cadaster identification number and the proper identification number generated automatically at inserting the record. The listed monuments are further documented with a registry number from the Central national list of monuments.

3.2.2. Unit location

This is described by blocks of administrative determination of the location, (state, region, district, basic territorial unit, municipality, etc.); historic (territorial, administration, Church units) and geographical determination; address data; cartographic data; cadastre data and land unit price data.

There are collected also data concerning changes of individual characteristics, e.g. administration units and recorded in a chronological way.

3.3.3. Property and juridical data

Here are summarised proprietor data, user data, historical review of proprietors, limits of use rights. The latter are concerned not only cultural heritage limits but also hygiene, basic building regulations (building barrier, land covering parameters), ecological protection and others.

3.3.4. Archive references

This block comprises administrative documentation of cultural heritage unit protection (inventory data, listing procedure data and proclamation); graphic documentation data (drawings, design plans, sketches, iconography and maps); text documentation data (bibliography, archaeological and other survey reports, findings, archives material and sources); ethnology documentation; environmental documentation, and monitoring reports.

3.3.5. Technical description and condition state

It includes blocks of basic data (mostly for buildings - urban situation, layout, structural system, building materials, vertical division, built or surface areas, volume); technical description of elements (in individual stories including quantification); physical condition of elements; utility equipment and supply network; arrangement of the lot.

3.3.6. Unit function

This part embodies blocks of data on contemporary function of the unit and original functional use. It contains data on functional composition of a unit, rental data, urban economy data associated with the unit.

3.3.7. Historical data

They group blocks on architectural history of the unit, relation to historical events and personalities and the unit's own chronicle. Special attention is paid to changes and architectural, conservatory or remedy interventions with evaluation of results.

3.3.8. Cultural heritage evaluation and protection measures

This block appraises the relevant features according to individual elements, stories, as well as the entire unit. It serves not only to describe the listed monuments but it is used, moreover, for inventory of cultural heritage aspects of other objects and sites. In this respect, for example, a methodology of Zuzana and

Jiří Syrový (1997) [3] is used for rural architecture. An illustration of such a survey is presented in Fig. 1 for a part of a World Heritage City of Telč suburb.

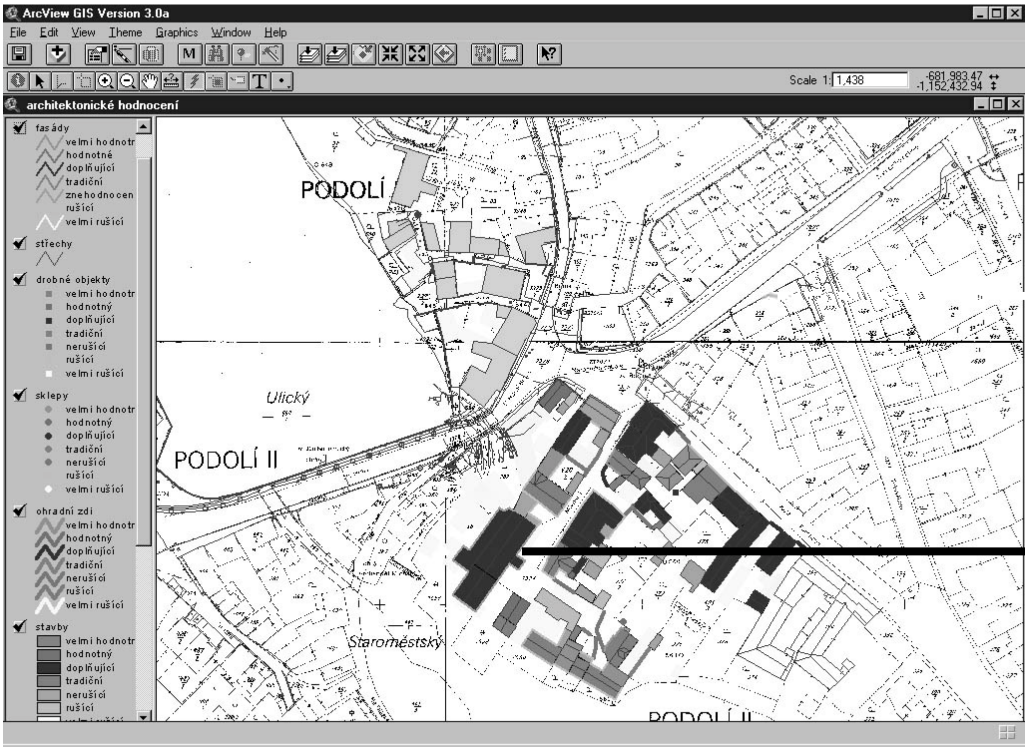

Figure 1 Architectural value evaluation layer of facades, roofs, auxiliary objects, cellars, fencing walls and buildings

4. STRUCTURE OF THE DOCUMENTATION SYSTEM

The basic structural chart of the documentation system is shown in Fig. 2. It is composed of three layers of data structures and programs. Data structures contain the main database and files of digital maps. Raster maps are stored as bitmaps (*bmp*), vector data use a special format. Export and import of vector maps is possible in formats *igs* or *dxf*. The map server is used to facilitate an access of application programs to data and it is for a user totally transparent. Application programs ensure the map management, the GIS function and the access to databases. First package serves for creation, up-to-dating and presentation of vector or raster maps.

A possibility of creation and actualisation of data by users on the place is particularly stressed because the system is aimed primarily for management and decision making in historic cities and must be easily up-to-dated. GIS programmes enable a search in databases in relation to map entities and vice versa, as well as graphical interpretation of the found data. The system is also intended to be accessible via Internet for public dissemination of open data and facilitation of the access to remote databases.

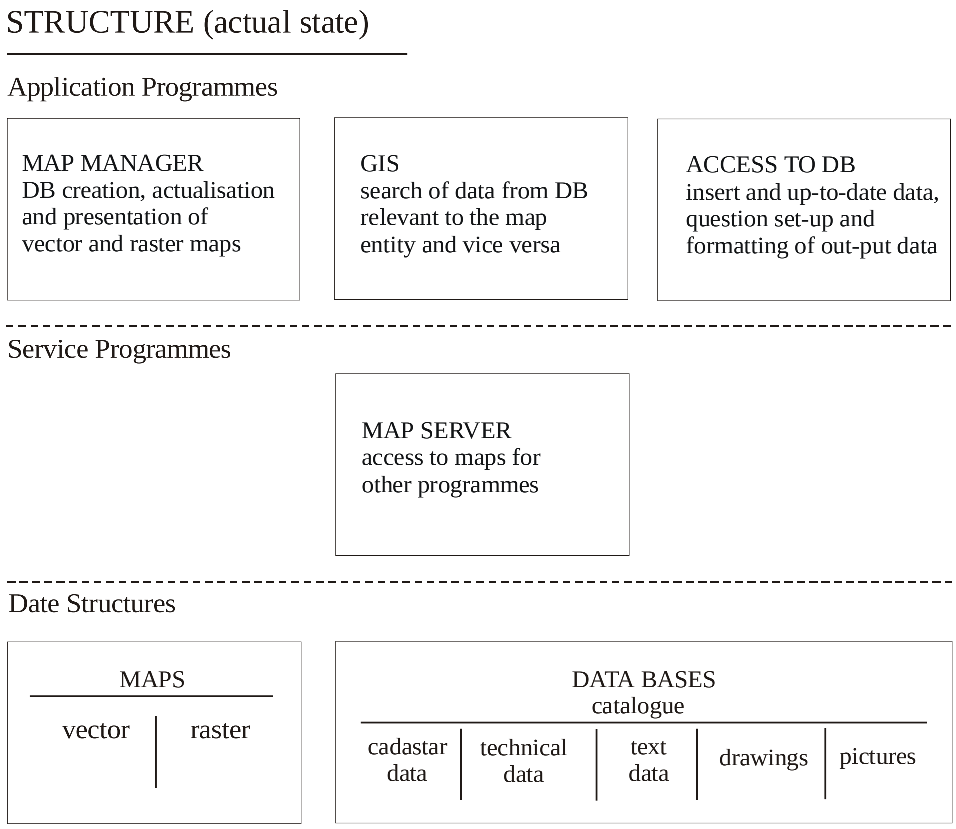

Figure 2 Structure of the documentation system

5. INTEGRATED CULTURAL AND NATURAL HERITAGE RECORDING IN A MICRO-REGION

In the framework of accompanying actions to the Follow-up program in the World Heritage City of Telč supported by the Council of Europe, there was elaborated a GIS based recording of a complex natural and built heritage over a broader territory and in context with historic settlements. This pilot project resulted in a rich database of vectors of collected characteristic data from more than 6000 land elements in the selected area of 230 km squared and prepared a good foundation for continuation in the development of appropriate tools for management of a cultural heritage landscape, [4].

The term micro-region is used here for a territory with significant interior, historic, cultural, functional, social and spatial links, in which the state governmental management is usually ensured by the historic town under consideration. The above mentioned documentation system is adopted in the micro-region, too.

The system enables creating of thematic maps and evaluation of development impact on the functional and conservation characteristics of the territory. An example of such a map is presented in Fig. 3.

SCALE 1:10 000
AREA OF 230 SQUARE KM
GEOSYSTEM VECTORS

- geology
- terrain
- climate
- humidity
- soils
- MAPS
- land use
- limits of utilisation (protection of nature, water, architecture, landscape, scenic and residential)
- landscape potential
- ecological stability
- AREA MAP UNITS
- 1700 area elements

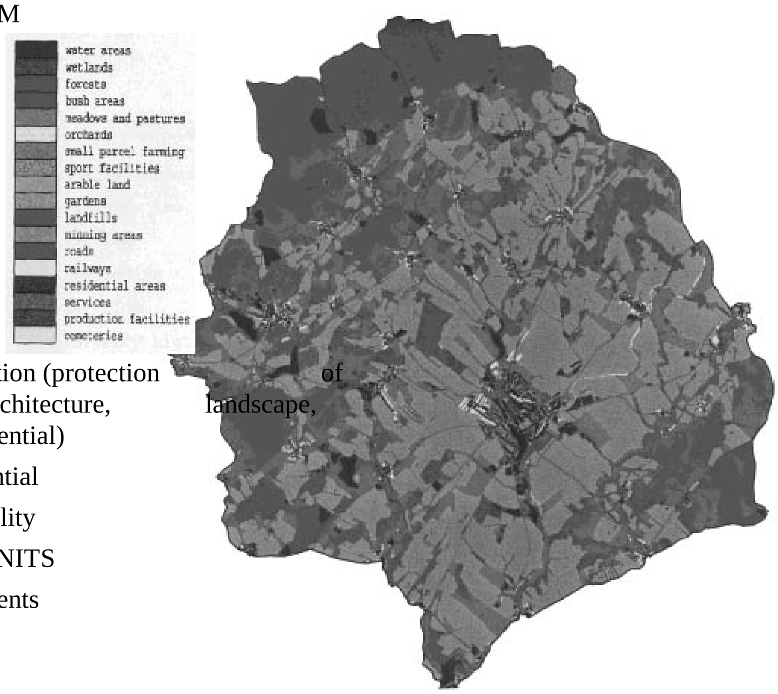

Figure 3 Land use map of Telč micro-region

6. CONCLUSION

The above described documentation system has been designed with a strong emphasis on applicability in historic villages and cities. Special layers of the GIS structure are therefore devoted to digital plans of architectural objects, presented in CAD format. It improves capability of the system for historic analyses as well as designing of changes in individual objects and easy visualisation of impacts, 3D modelling etc. Figure 4 presents CAD picture of a part of the Telč historic core.

Technologies and computing devices used for the system exhibit user friendly characteristics and enable application even in poorly developed environment. The possibility of creation of individually tailored outputs is highly appreciated by cultural heritage officers and city clerks as a suitable tool for planning and management in historic sites, ([5] with further references).

Figure 4 Detail plan layers of the documentation system (CAD) – a part of the Telč square

7. ACKNOWLEDGEMENT

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DEFORMATION OBSERVATIONS AT THE CHURCH OF SERGIOS AND BAKCHOS BY PHOTOGRAMMETRIC TOOLS

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Abstract

The church of Sergios and Bakchos / Istanbul was constructed in the beginning of the 6th century (built by Justinian 527-536). Since the year 1504 until today the building is used as a mosque. The new name is Kücük Ayasofya Camii. The cultural importance of the building doesn't correspond with the very poor shape: the dilapidation is documented by many photos, which have been taken since 1900 in different time intervals. Since 1979 a lot of high precision photogrammetric and geodetic surveys have been carried out to monitor the deformations of the building.
The deformation analysis presented in this paper is exemplary focussed at the following four points:
a) the inclined positions of the eight pillars of the central octogon; we have found a maximum in the inclination of the pillars of about 40 cm at the height of the pillars of about 9 m,
b) the inclination of the whole apsis; which is about 40 cm of the wall in front, the height of the wall is about 10 m,
c) some of the big open cracks; we have observed an opening of the cracks of about 1 cm between the years 1979 and 1998,
d) the new damages found after the Gölcük-earthquake of August 17th,1999 (magnitude 7,8).

1. Introduction

At first we want to look at the situation outside of the building, figure 1 and 2, where a lot of changes are illustrated. We start with the situation of the "new" entrance in the west built in ottoman times (1504), top of figure 1.

figure 1
West and south of the building,
ottoman entrance, situation of the railway

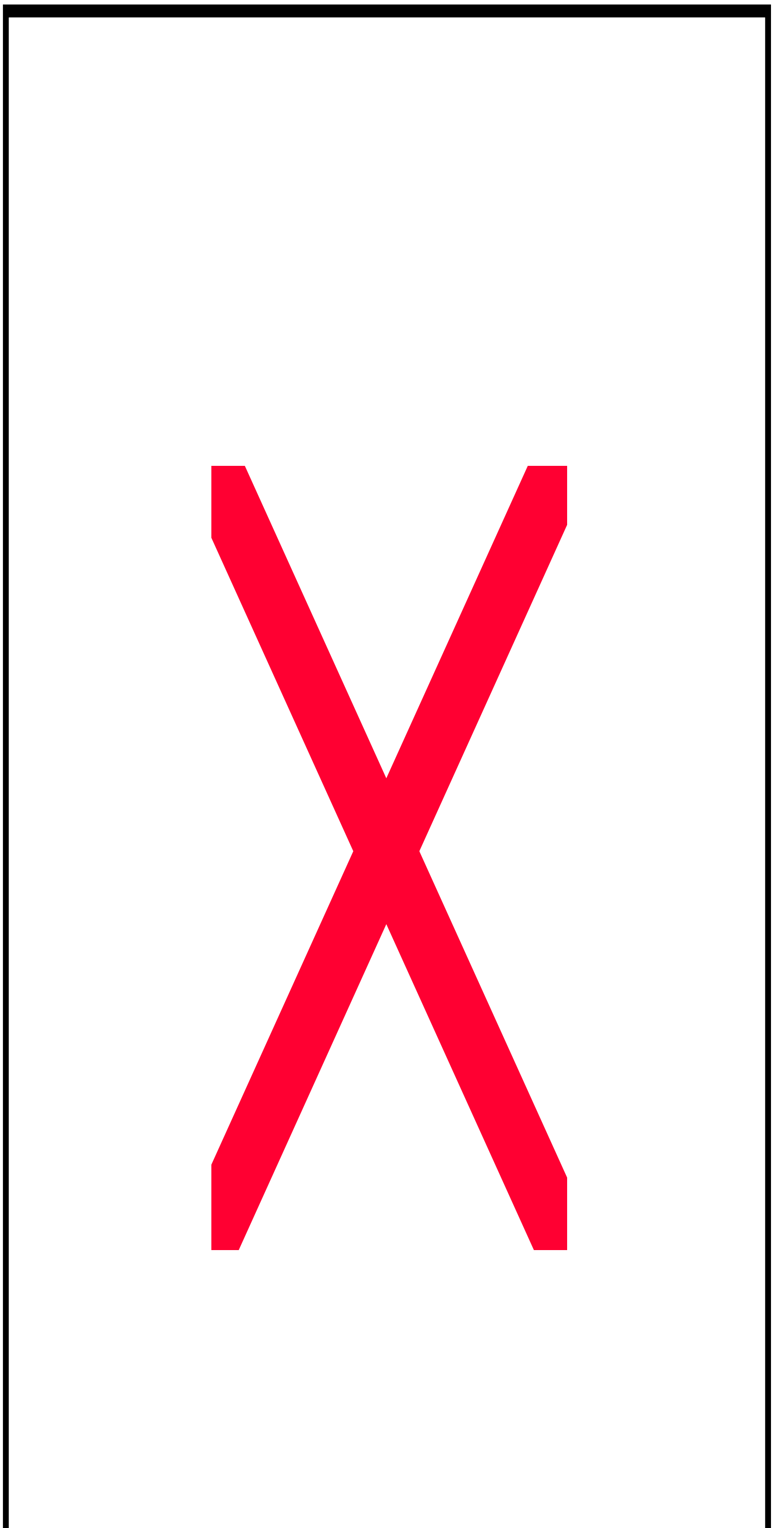

Figure 2
Photomosaic of the exterior wall in the north

Let's realize the situation in the south, bottom of figure 1: In the years 1870/71 the main railway at the European side of Istanbul was built in a distance of only a few meters. Today the frequency of the suburb trains is about 10 minutes. Also the well known Balkan express and some heavy freight trains are using the route. We think, there are a lot of defects in the building resulting on the vibrations generated by the trains, at first directly and second as a result of the local differential settlements of the underground beneath the railway due to soil densification. It is supposed that every train is a kind of vibrator to compress the underground which influences also the situation of the water in the underground. The exterior wall in the south of the building was renewed in 1877, a short time later than the use of the railway started.

At the wall in the north, figure 2, we see a lot of different materials and building structures made at different times. This patchwork tells us, that the condition of the building reflects things like earthquakes, vibrations, moisture and so on.

In the last years many contributions about Kücük Ayasofya have been published, we look only on two of them. Svenshon and Stichel [2] present and found the thesis, that the whole narthex is not from the original building, that the doubling of the southern wall is a repairing work because of static problems and that some big parts of the northern wall are also results of repairing works. The origin of all those parts of the building is not from the times of Justinian. Only the central octogon, most parts of the apsis and some parts of the northern wall are from the original beginning of the building.

Özsen and Bayram [1] present and discuss the structure of the dome over the central octogon. The dome is composed of sixteen elements. The eight elements with the integrated windows are ellipsoidic curved only in one direction from the border to the central point of the dome. The other eight elements starting at the pillars are bidirectional curved, first to the central point like the others and second in the transverse direction to get the needed static power, see also figure 3 for information.

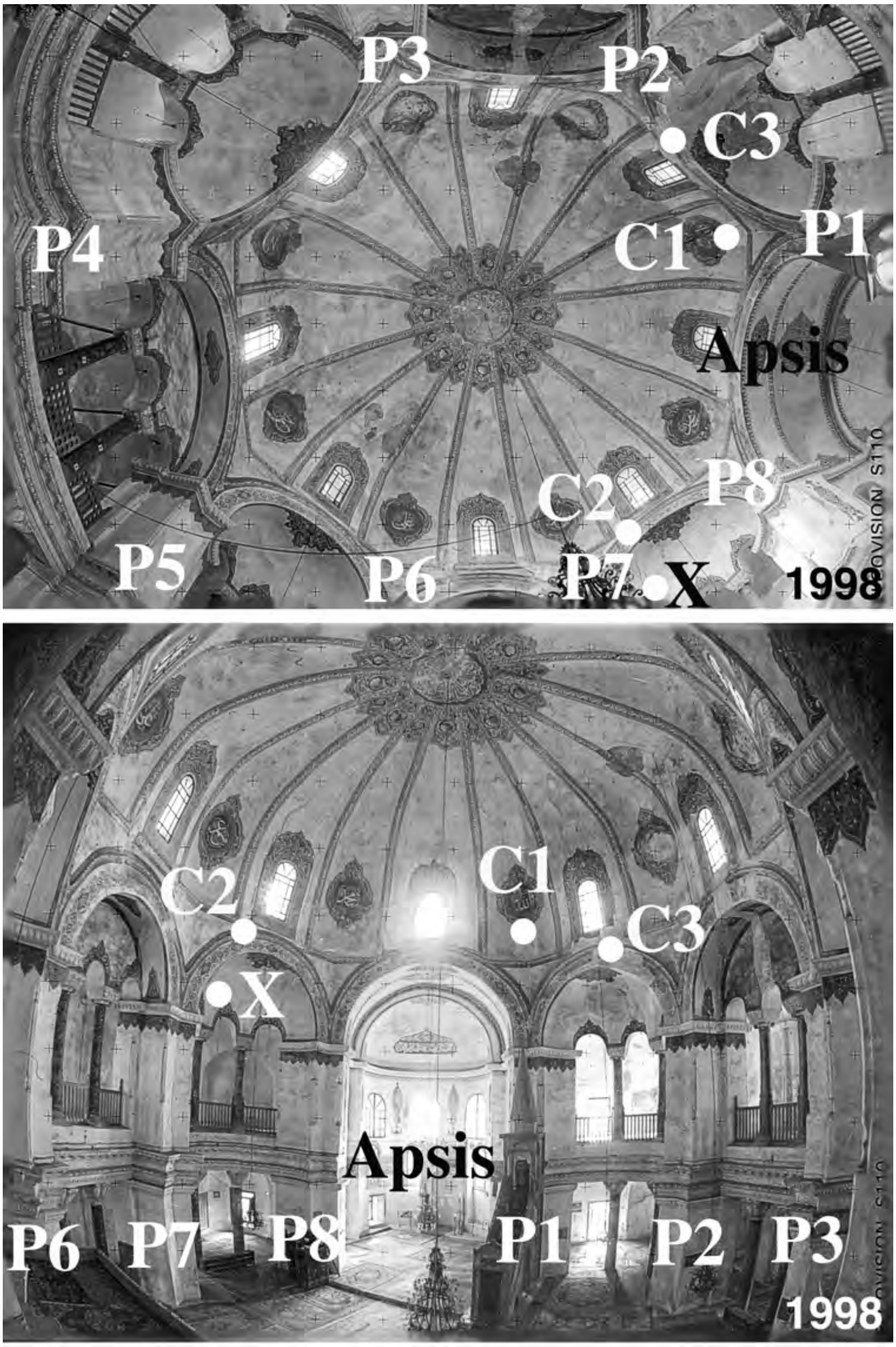

figure 3
Vertical view to the dome and horizontal view to the
central octogonal room with the apsis

2. Inclinations of the pillars and the apsis

Photogrammetric surveys have been carried out to get some information about the inclinations of the eight pillars. Plumblines with some marked points were mounted on the tops of the pillars, photogrammetric images were taken and analysed.

The inner side of the apsis was measured at first to derive a CAD model. We determined an inclination of the whole apsis. To proof this, we took some photogrammetric images outside the apsis to control the measurements with an exactly levelled horizon. The inclinations of the pillars and the front wall of the apsis (two points) are presented in figure 4.

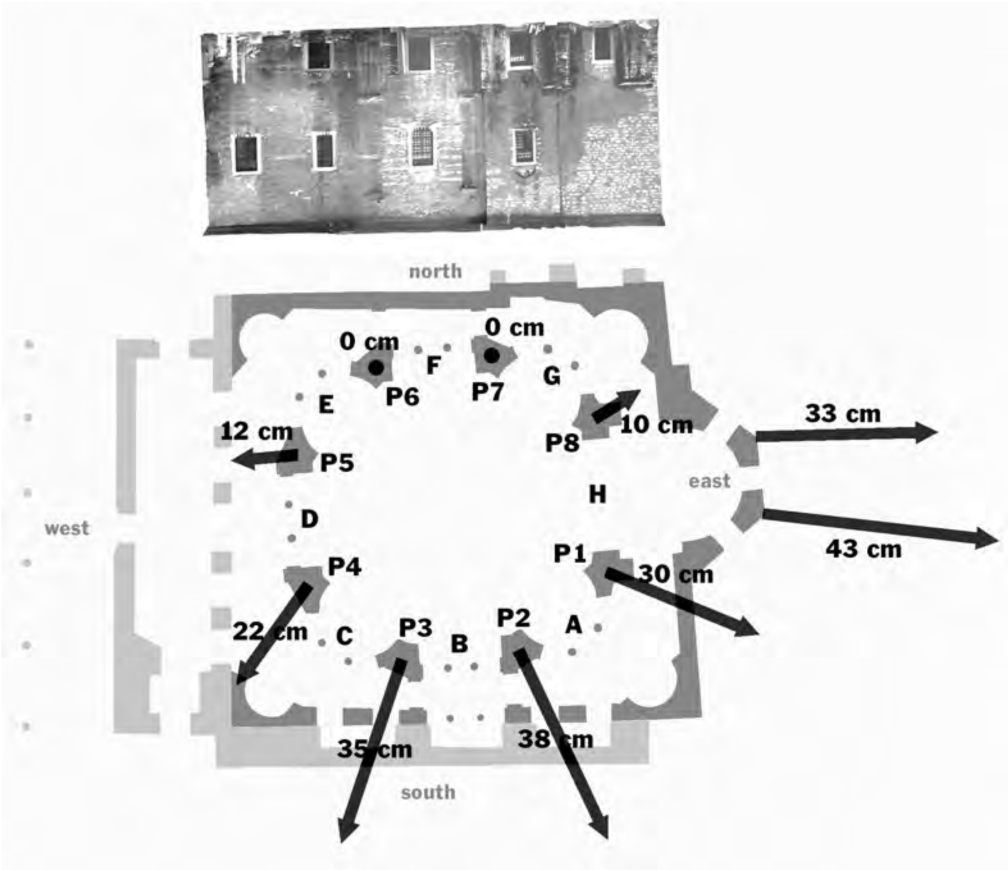

figure 4
Inclinations of the pillars and the front wall of the apsis
Height of the pillars: about 9 m, height of the front wall of the apsis: about 10 m

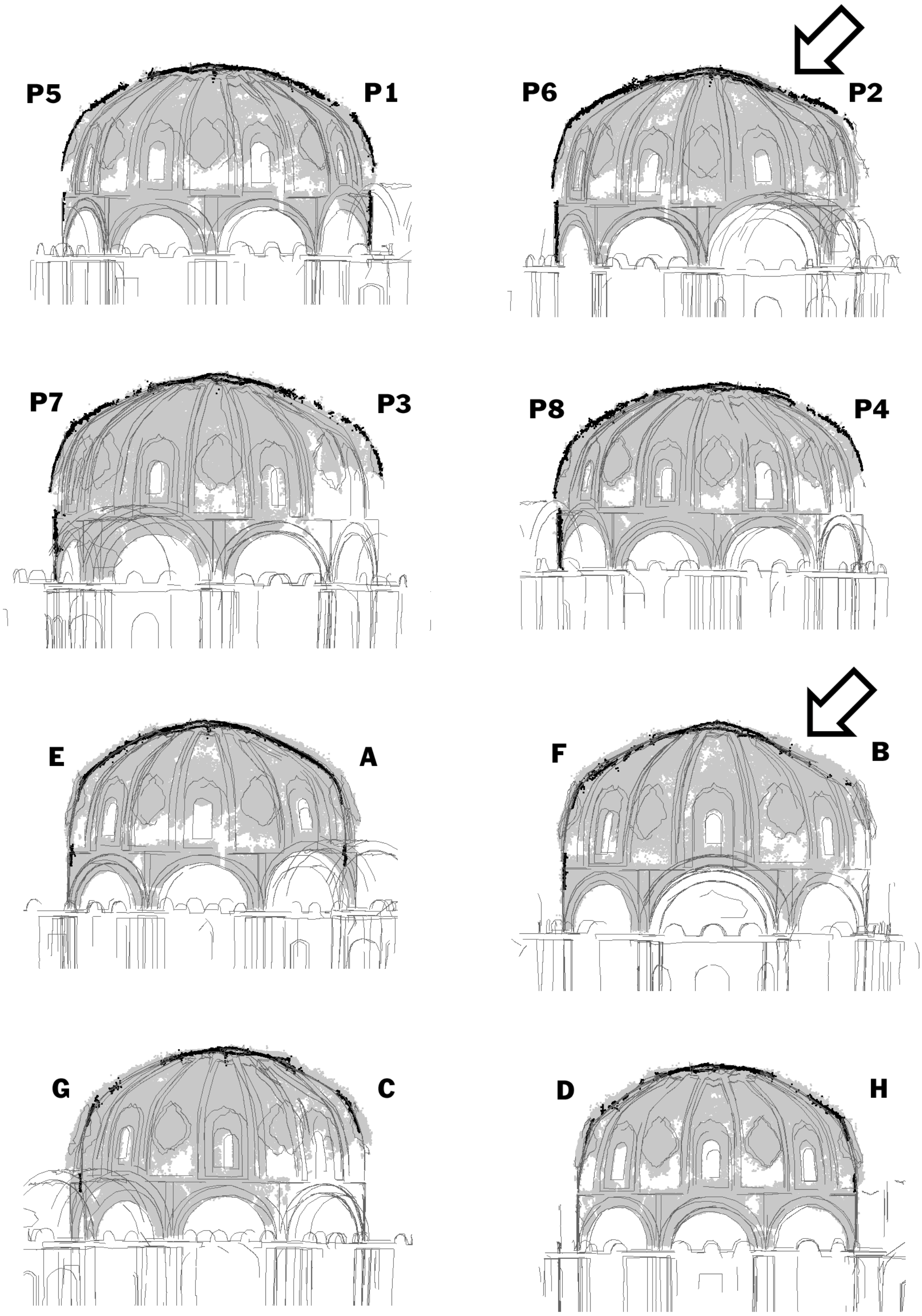

Figure 5
Profiles of the dome,
deformations

We think the systematic inclinations of the pillars are reasons for the deformations of the dome, the ideal form is disturbed in the southern part, neighboured to the pillars P1/P2/P3. The dome shows a kind of bump. The shape analysis of the dome was done with measurements of photogrammetric images in an automatic way. First analyses have been made by looking at some profiles between the pillars P1-P8 and also between the points A-H signed in figure 4. In figure 5 the profiles show the deformation of the dome. Also big parts of the southern exterior wall are inclined in the same direction as the pillars, which was apparently the reason to build a second wall in the south, figure 4.

3. Observed and analyzed cracks

Next we want to look at some selected cracks (C1,C2,C3) in the building. The locations are marked in figure 3. The crack C1 can be identified in some prepared images taken in the last 100 years, figure 8. The first image was taken between 1885 and 1908. At this time the mosque was renovated. In the years later the crack appeared. We know that the drum of the dome was plastered in the year 1955 and later on the same crack was visible at the outside. So we can say that crack C1 occured in the last 100 years, the width of the crack is about 2 cm, compare figure 6. We think that crack C1 is very important because the pillar P1 is affected.

Figure 6
Width of the crack C1

Crack C2 is starting in the dome and runs through all the components of the building ending in the outer wall in the north, for illustration look at figure 7.
In the region of crack C2 we have measured and analyzed photogrammetric images from the years 1979 and 1998. We found an opening of the crack of about 1 cm for the period of 20 years, figures 9 and 10.
The photos we have taken in spring 2000 after the earthquake from August 17th, 1999 show another opening of the same crack at different other locations, also outdoor in the northern wall.

Let’s look at figure 11: in the year 1968 crack C3 was in a symmetric situation to crack C2. At the moment crack C3 is plastered. We know from other images, that it was also plastered in the year 1979. Therefore we can conclude, that the movement at crack C2 is bigger than the movement at crack C3.

Figure 7
Crack C2 inside and outside
after the earthquake from August 17th, 1999

At last we want to show some damages of the earthquake from 1999. Figures 12 and 13 show some sort of “normal” damages in the mosque. Figure 13 is very interesting in another way: we have analyzed some older images from the same situation and it is obvious, that the contour of the broken part is also found in the image taken before 1908. Therefore we can say, the damage from the year 1999 is a result of a weak structure from older times. As a consequence we can formulate the thesis, that most of the weak points of the building are very old and the things which happen - like earthquakes, vibration of railways, moisture etc. - only show the inherent damages.

4. Conclusions

Above we have presented two principal different kinds of deformations: At first the systematic deformations of the pillars especially in the southern part of the building and the resulting deformations in the neighbouring parts of the dome,

also the resulting large inclinations of the whole older southern wall. We assume all those deformations to be old, may be they exist since the beginning of the building. This thesis should be discussed.
The other kind of deformations is newer, the big cracks are active but not in a good way, we think. There are movements of big parts of the building particularly in the eastern and southern regions, may be there are weak points in the foundations of the building in the underground, may be there are settlements of the underground beneath the railway .
We hope that the presented deformations will trigger motivations to do the needed works to stabilize the Kücük Ayasofya Camii, one of the most important byzantine buildings of Istanbul and an absolute highlight in the history of European architecture.

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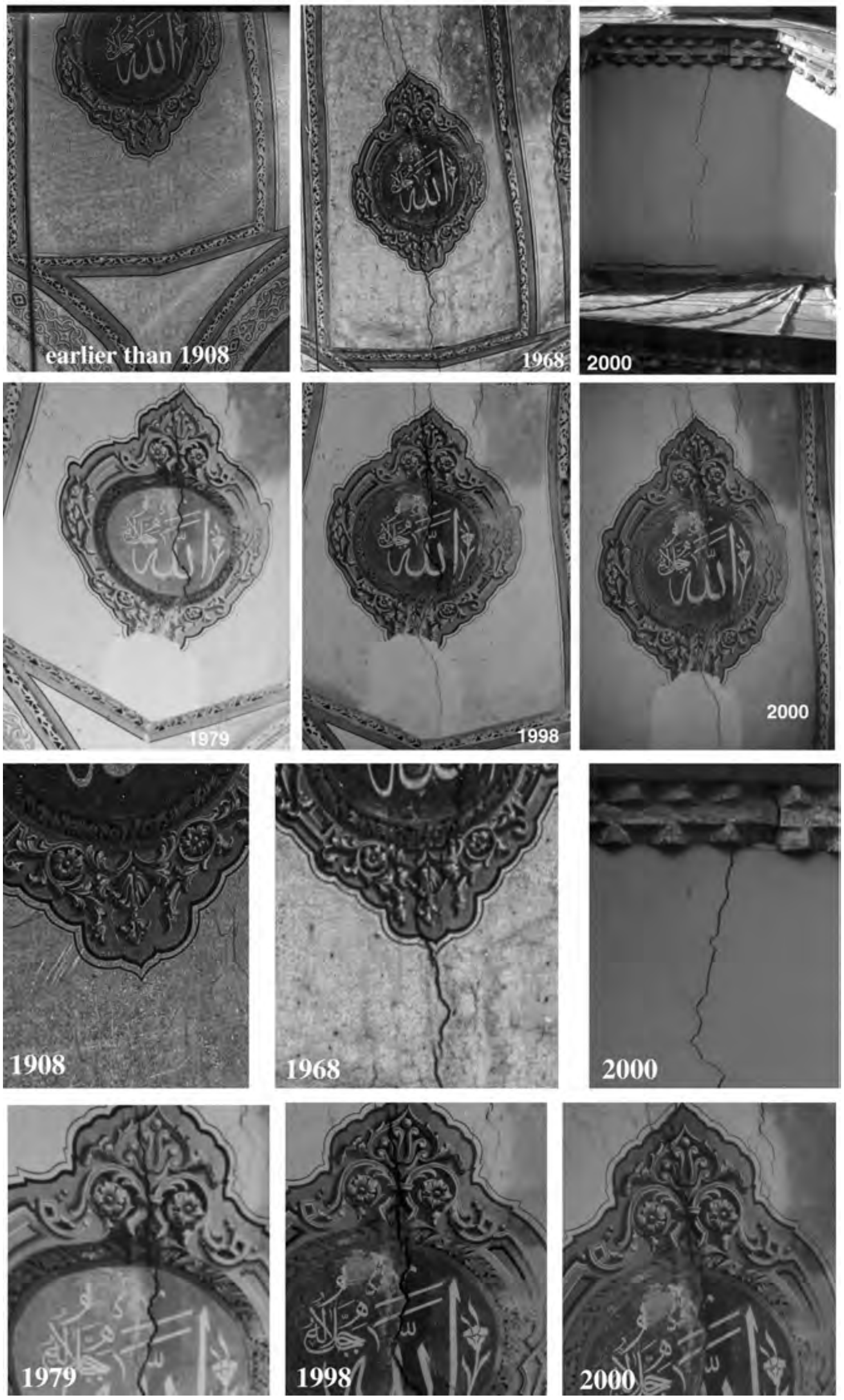

Figure 8
Crack C1 inside and outside the building,
images taken in the last 100 years

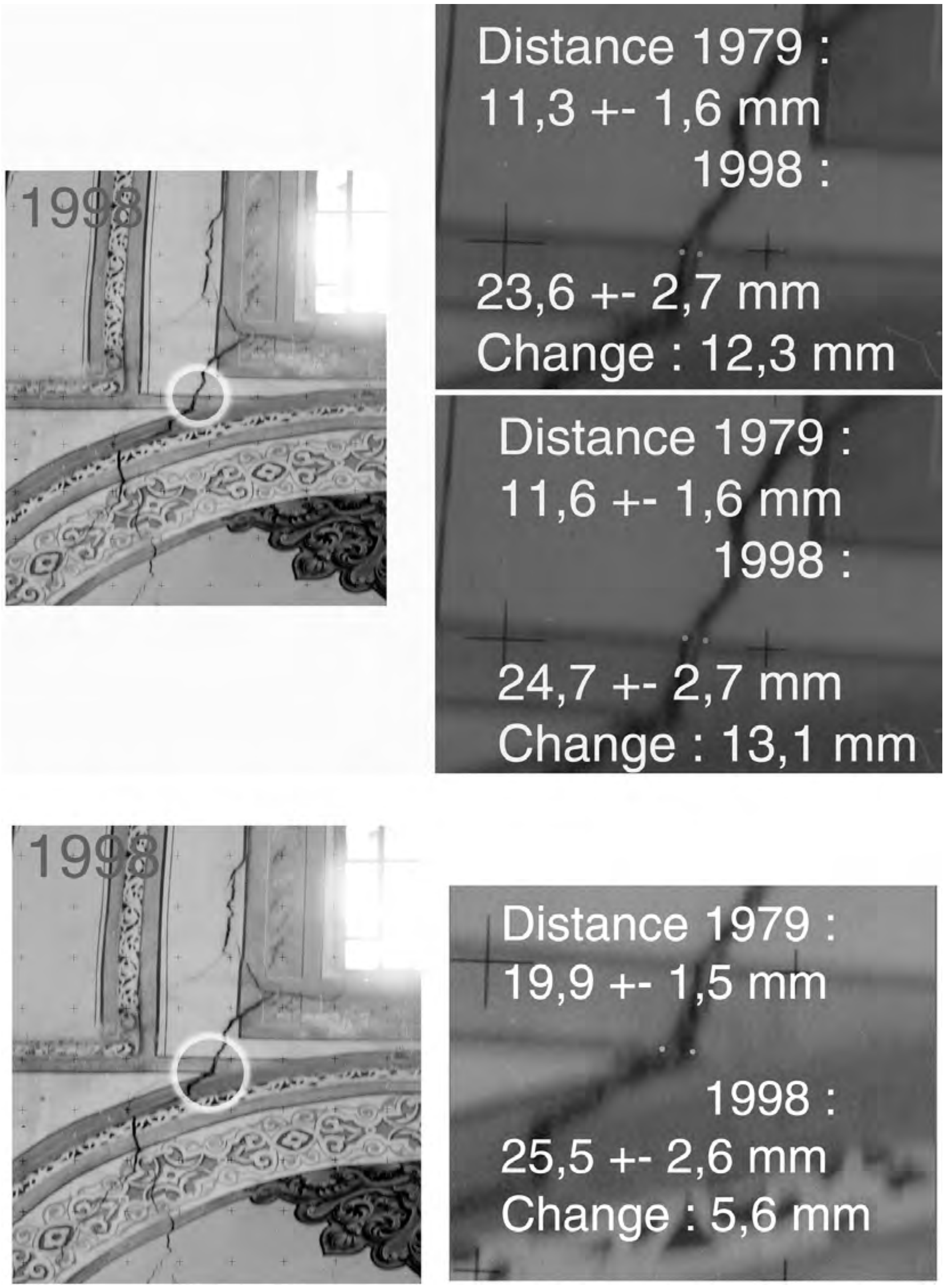

Figure 9
Change of the width of crack C2

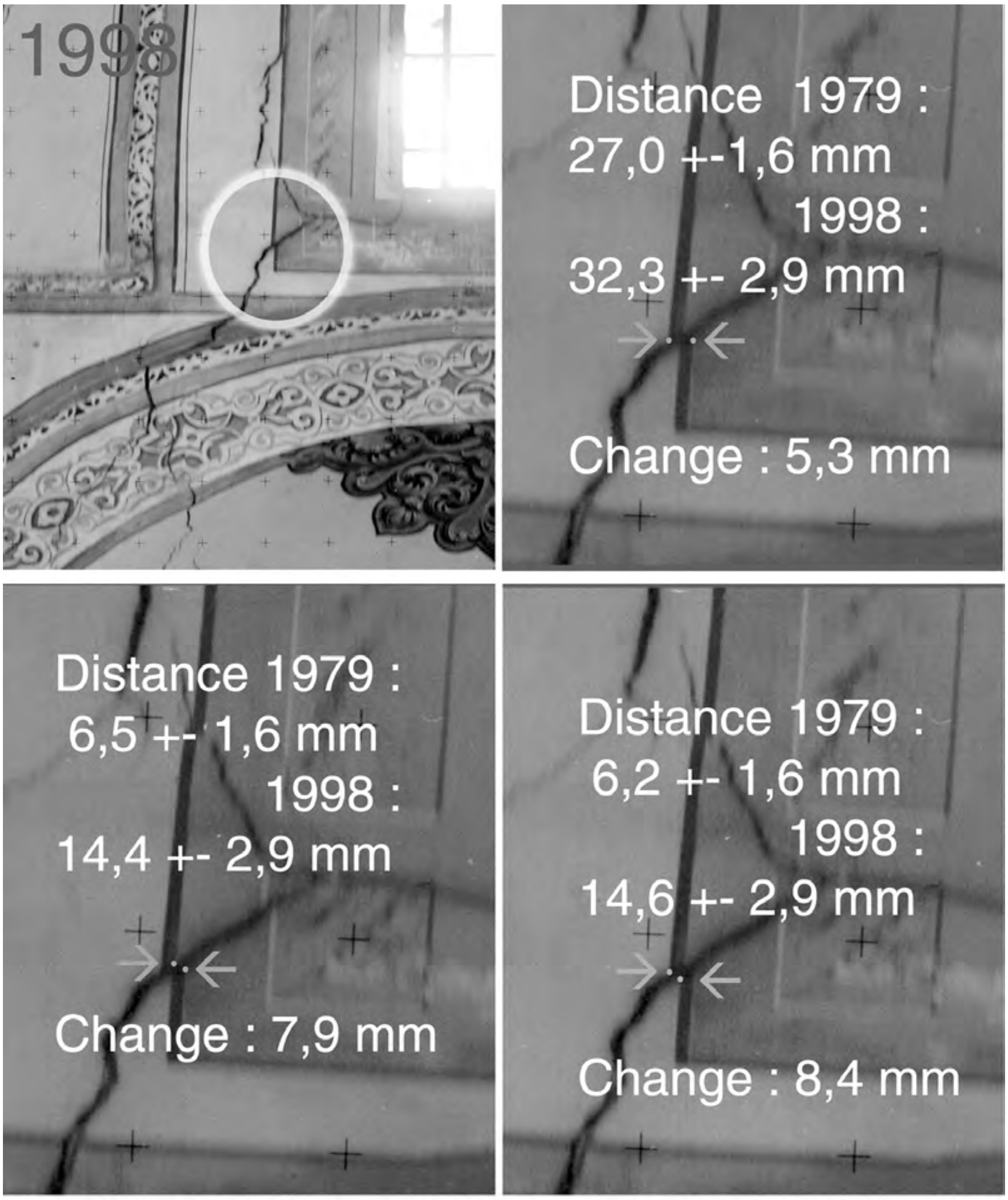

Figure 10
Change of the width of crack C2

Figure 11
Crack C3, images of the years 1968 and 2000
(The image of the year 1968 is rectified to the image of the year 2000.)

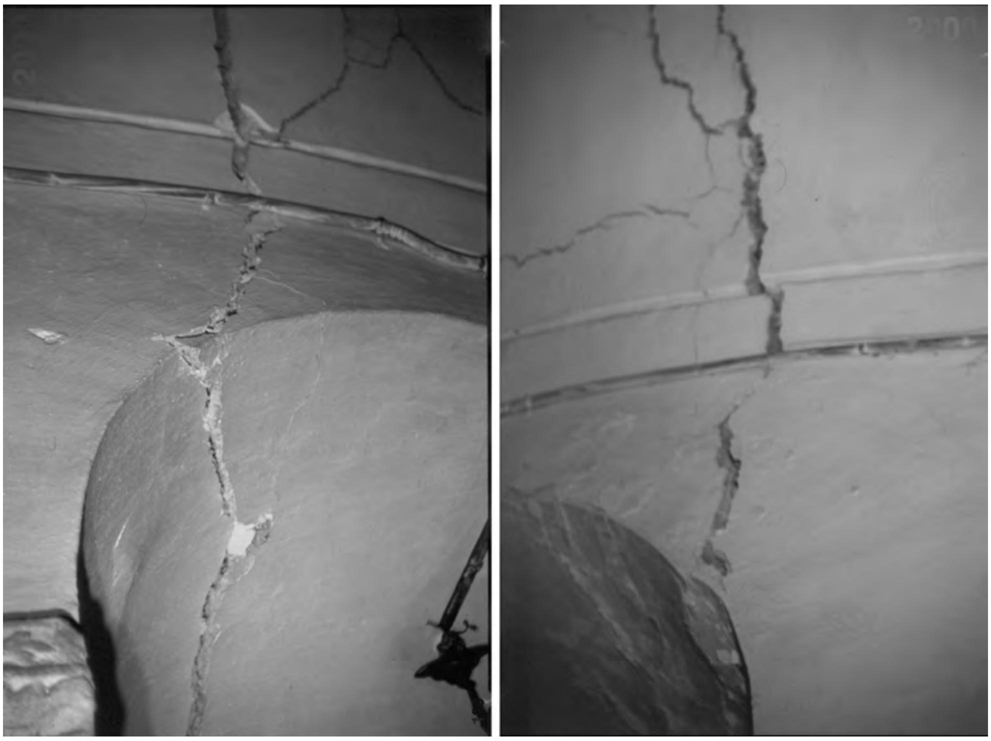

Figure 12
Some damages of the earthquake from August 17th, 1999

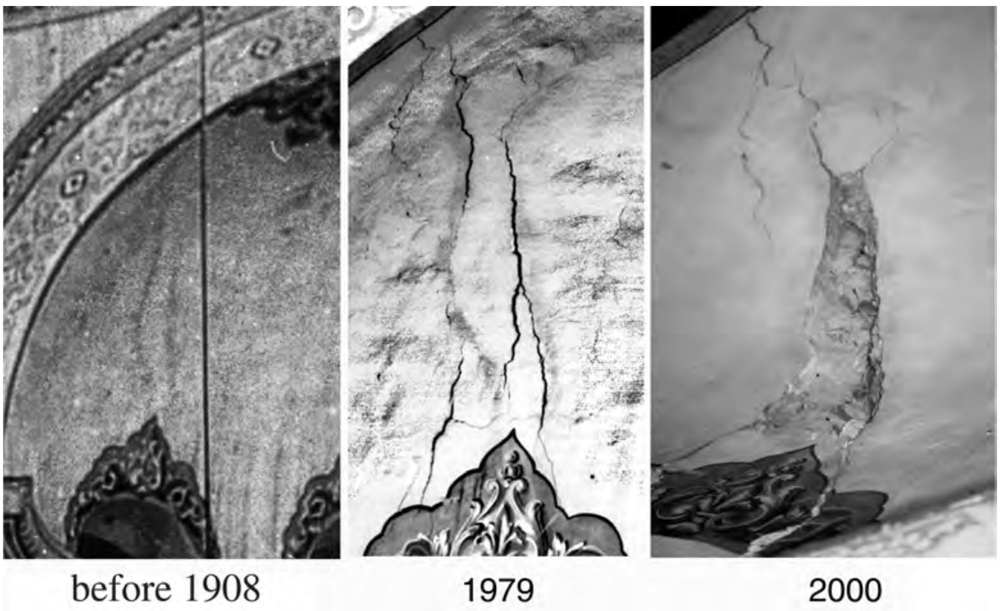

Figure 13
Some damages of the earthquake from August 17th, 1999,
(point X in figure) same situation 1979 and before 1908

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NURBS MODELLING FOR THE CONSERVATION OF ANCIENT BUILDINGS

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ABSTRACT

The use of recent software which integrates the use of NURBS surfaces enables us to solve a construction's cognitive track in a completely original way, combining traditional methods of surveying with the possibilities offered by modern IT tools. This study concerns a tomb (IV – III century BC), situated in the ancient centre of Egnazia, and an underground olive-mill (XVI century AD) entirely dug out of rock and with a very irregular volumetric development. The need to obtain a tri-dimensional restitution suggested the adoption of NURBS modelling for the construction of a geometric model which was completely flexible. This was generated by starting from the individuation of a series of curve-sections (*Spline*) carried out *in situ* by means of tri-lateration methods and orthogonal Cartesian co-ordinates. The NURBS model obtained from the union of the single sections. It was perfectly manageable even by calculators with medium-level hardware which were built without resorting to polygonal geometry. Afterwards it was possible to add details of the areas of major interest, and so extrapolate the bi-dimensional processed data necessary for the research in progress.

1. INTRODUCTION

The use of NURBS (Non Uniform Beta-Spline) surfaces for the tri-dimensional modelling has been, until recent times, a precious instrument for research in the biomedical field and for the virtual scenes, especially in the cinema. The evolution of these modelling techniques, their proven reliability and the capacity to check the co-ordinates would suggest to try them out in the fields of conservation and restoration. The three-dimensional reconstruction of a complex structure and its representation by means of NURBS surfaces offers an indispensable base for the understanding and the analysis of the state of conservation of a construction. In fact, this technique is a natural evolution of the traditional bi-dimensional survey.

2. NURBS MODELLING

Tri-dimensional modelling is used to represent objects in 3D space. The most common modeller is the polygonal one. In this type of application, the surfaces are defined like a set of small squares or triangles. The main advantages of polygonal modelling lie in the number of different types of surfaces that can be defined and the ease of converting the CAD/CAM data. The polygons find their ideal field of application in the modelling of objects that do not change shape or that are made up of "hard surfaces" with sharp edges. For the creation of objects with an "organic shape", the number of vertexes necessary for an acceptable definition could be more difficult to manage in comparison with other modelling methods. For this reason, over the last few years, especially in the aerospace and biomedical research fields, the surface modellers that make use of so-called "spline curves" has become more widespread. The most interesting aspect of the use of "splines" lies in the possibility of defining a complex surface by using very few points and in their description by mathematical functions, which are not difficult in calculation terms.

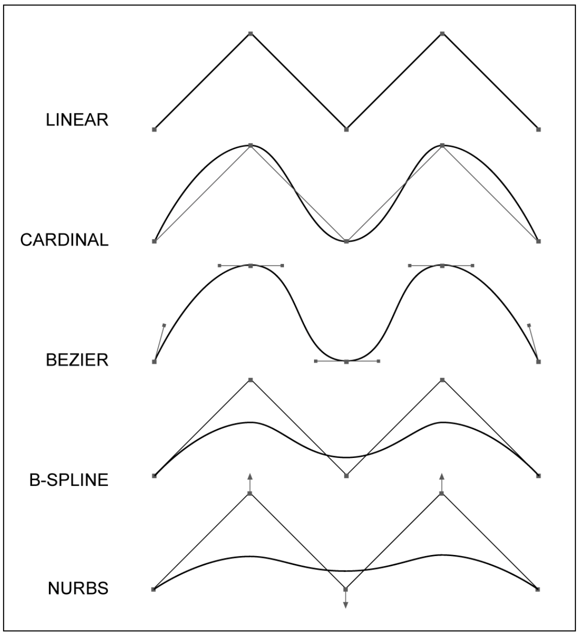

Figure 1

Five types of spline are most common [9], and each of these differs from the other from the point of view of the management of the control points (Figure 1).

- *Linear*: appears as a series of lines that connect the control points.
- *Cardinal*: this curve goes through the control points and allows each point to have a tangency check.
- *Bezier*: this curve is similar to those used in the vector graph programmes; in this case the curve goes through every control point and allows each to have two tangency points.
- *B-Spline*: this type of curve rarely goes through the control points (in this case the points are called "knots").

- *NURBS* work in a similar way to the B-spline, but each knot can have its own weight. A NURBS surface can also be generated by the other four types of curve above described.

The NURBS surfaces and curves do not exist in the field of the traditional design. They were expressly conceived for computer-assisted tri-dimensional modelling. Curves and surfaces represent the external profile or shape of an object in 3D space.

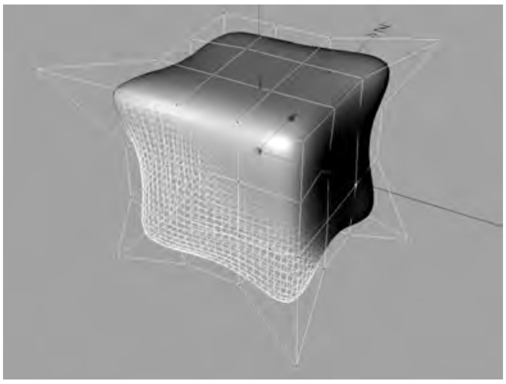

Figure 2

NURBS is the acronym of *Non-Uniform Rational B-Splines*, where:

Non-Uniform indicates that the range of the checks on the vertexes can be very variable. This is of extreme importance in the modelling of irregular surfaces. Rational means that the equation used to represent the curve or the surface is expressed as the ratio between two polynomials, rather than a simple sum of polynomials. The rational equation describes a model or some important curves or surfaces in the best way possible, especially conic sections or spheres.

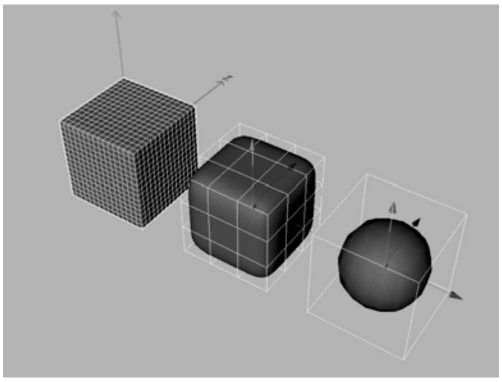

Figure 3

The non-uniform property of NURBS offers an important advantage in the modelling process: since they are generated by mathematical expressions, the objects can be built in a parametric mode, in addition to the normal possibilities of manipulation of 3D geometry.

More specifically, a set of nodes determines the level of influence of each of the control vertexes (CVs) belonging to the curve or the surface. Such nodes are invisible in 3D space and are manageable only in the modelling phase; however, their manipulation influences the appearance of the NURBS object (Figure 2). The software available on the market today implement the NURBS curve in very different ways and offer specific modifying tools depending on the philosophy of the work. For this study we used the *Maxon Cinema 4D v.6* packet, for the modelling and the rendering, which showed remarkable flexibility and reliability even in critical work conditions. The C4D Hyper NURBS, for example, use an algorithm to subdivide and round off an object interactively; this process is defined as *subdivision of surfaces* (Figure 3). With this technology it has been possible to create objects of an “organic shape”, like blocks of worked stone, in a

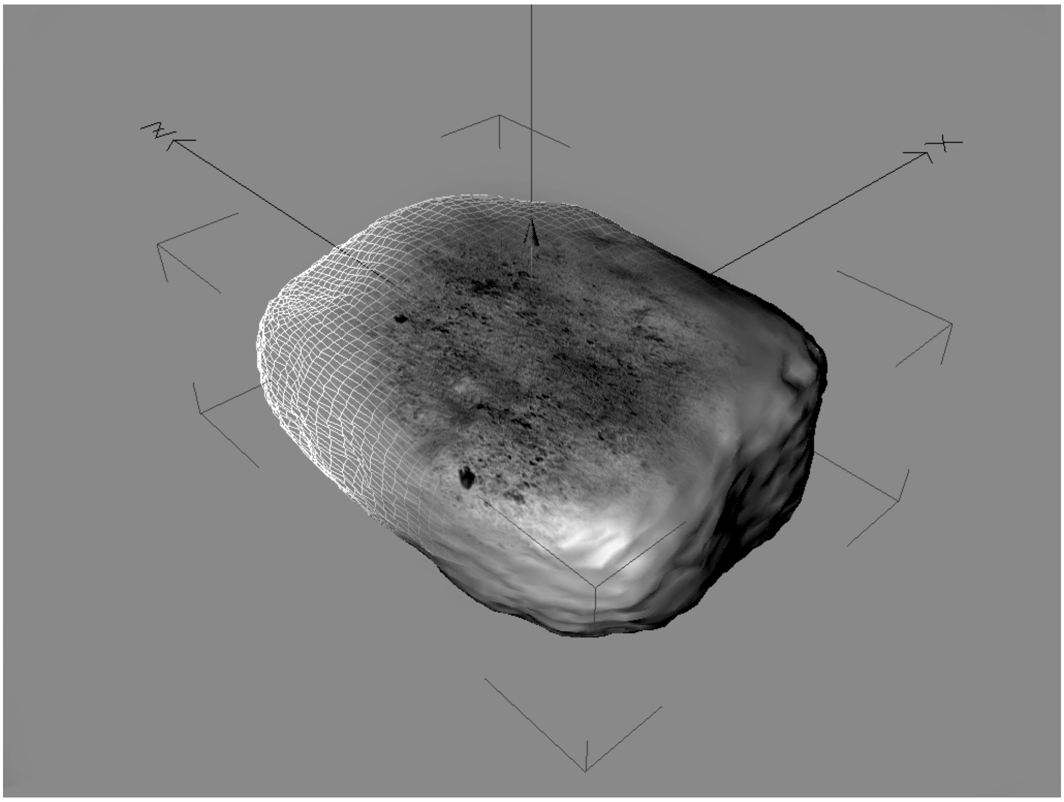

Figure 4

completely original and precise way, while keeping down the dimensions of the file (Figure 4). The instrument used in the most productive manner was the *Loft NURBS* (Figure 5): the splines are used here as profile-sections which define the surface development of an object. Two or more curves are connected in ordered

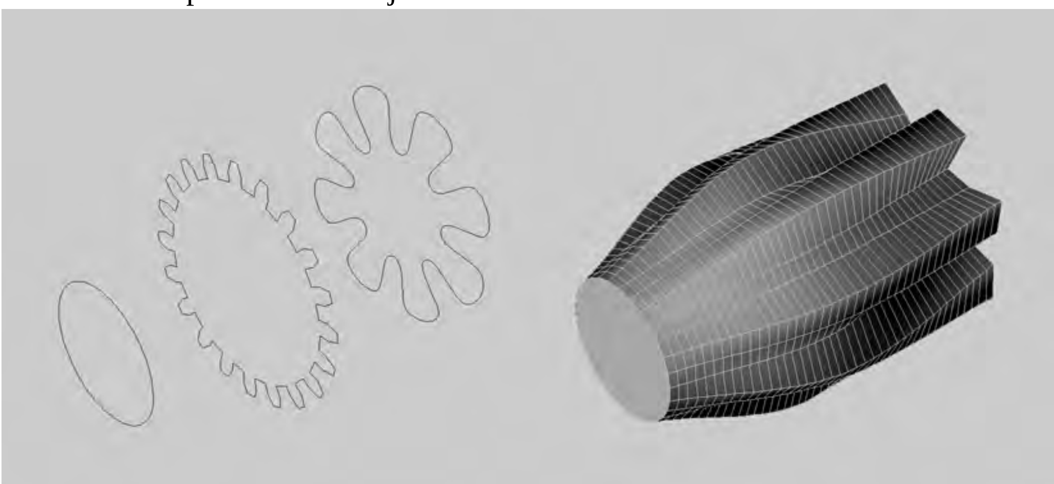

Figure 5

sequence and by means of interpolation the corresponding surface is obtained. Actually, the models drawn up for the reconstruction of the hypogeum olive-mill at Racale (Le) and for the reconstruction of the hypogeum tomb chamber in Egnazia (Br) are *Loft NURBS*. Their important characteristic is that the reliability of the computerized restitution is established by the degree of approximation with which the diverse curve-sections are carried out *in situ* and by the level of subdivision used for the definition of the details. Every model can be described with arbitrary levels of precision, but at any moment it can be enriched with details by simply adding more curves or points on the existing curves. The degree of precision of the representation is, therefore, unlimited. It derives from the specific topic and from the scale factor of the restitution. Then, NURBS surfaces are an ideal instrument for the creation of 3D archives of complex objects and for architectural survey. Their application is even more profitable in the reconstruction of archaeological sites (*Virtual Archaeology*) [4, 8] and ancient monuments, rarely defined by "hard" surfaces. It is, on the other hand, certain that the collection of data acquired about a specific archaeological site is in continuous evolution, in relationship with the new data from the dig. The NURBS surfaces also are useful in the case of rendering or animation phases. The images presented here (Figures 8-10) are a meaningful example of the level of realism achieved, both in terms of modelling, and of visual impact given the advanced capacity of UVW mapping (Figure 6).

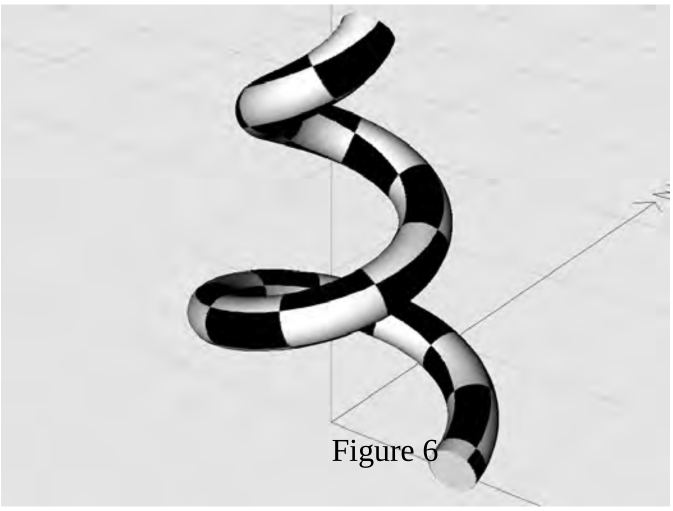

Figure 6

2.1. The Pilaster Tomb

The Pilaster (or Little Gate) Tomb [1, 3] is in Egnazia (Brindisi), a town situated on the Southern Adriatic coast of Apulia and were lived a native population called Messapians by the Greeks.

The chamber tombs date between the second half of the IV century and the II century BC [12]. There are not many of them in the region and they belong to the most important, "aristocratic" families. The tombs are always underground, have an entrance hall and one or more funeral chambers set out in various ways [5, 6]. The Pilaster Tomb belongs to that category of hypogeum tomb which has several funeral chambers. It has a rectangular vestibule (4.80x1.60m) with a NW/SE orientation, on the shorter sides of which two funeral chambers open. Chamber A on the south-east side is irregular in shape, measuring about 5.50x3.90m; it contains a central pilaster and a niche dug out of the central part of the wall opposite the entrance. The

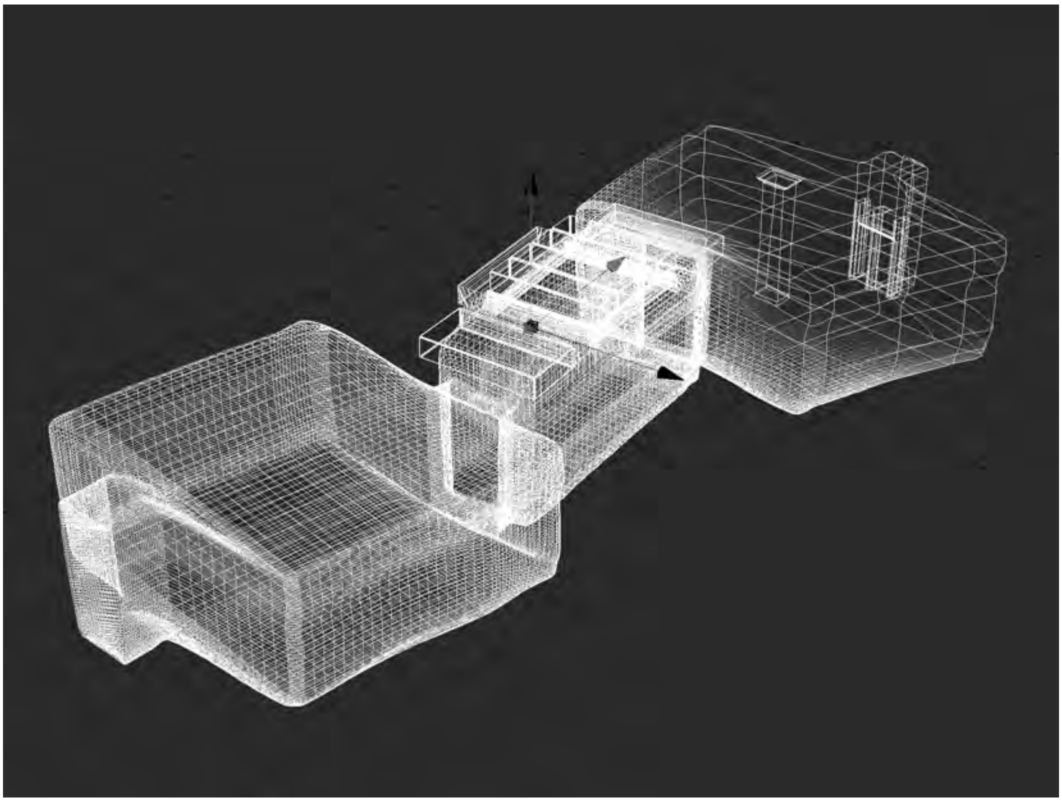

Figure 7

plan of the hypogeum can be compared with two other Messapic tombs: the first lies in Egnazia [3], while the second is in Rudiae [2]. The vestibule has a crowning cornice and is covered in large slabs. All the walls of the vestibule, of chamber A and part of chamber B are plastered and painted. As far as the “construction” of the tomb is concerned, the following phases are showed. First a deep rectangular pit, corresponding to the vestibule, has been dug out of the rock. Then, starting from the walls on the short sides of the pit the excavating was carried out from a “tunnel” to create the two funeral chambers; in this phase the rectangular pit was used to carry off the waste material from the excavations. The crowning cornice and the covering slabs in the vestibule were placed in position once the funeral chambers had been completed.

The survey carried out on the tomb brought to light a series of interesting details from a constructional point of view, which are difficult to spot at first glance. First we see a slight incline, towards the vestibule, of the floors of the two funeral chambers. This is the same gradient as that of the ceilings. The end walls of the two rooms show a certain perpendicularity in the horizontal floors, demonstrating therefore that the apparently “organic”, irregular development is subordinated to the functionality of the whole. This solution was adopted, probably, to guarantee that any water that had infiltrated into the hypogeum would flow towards the entrance, thus keeping the funeral rooms dry.

Figure 8

The 3D study also showed that the residual section of the rock thins out to a thickness of 30 cm in chamber A, so it was therefore necessary to guarantee more structural rigidity; the central pilaster placed in correspondence with this thinning is the architectural solution.

2.2. The Olive Mill in Racale (Lecce)

Underground and semi-interred olive-mills are a particular heritage in Salento (Apulia), connected to the age-old regional production of oil and to the ancient traditions of rural civilisation [10]. These "workplaces" were, from the XVI century on, used for the milling and pressing of olives to obtain the oil. Because of the unhealthy conditions that made these places unfit for working in and of the expanding industrialisation, the olive mills have been gradually neglected and abandoned from the first half of the XX century up to the present day. These places, with their machinery and tools, have become privileged objects of conservation and a living witness to the folk traditions. The Salento mills are prevalently hypogeum, that is, dug out of the calcarenite rock which is "Lecce stone", "carparo" or tufa. The characteristics of the constructions were studied carefully so as to carry out the various phases of the productive process in the best way and above all, to conserve the oil; in fact, the room temperature had to be warm and constant (18-20°C) which

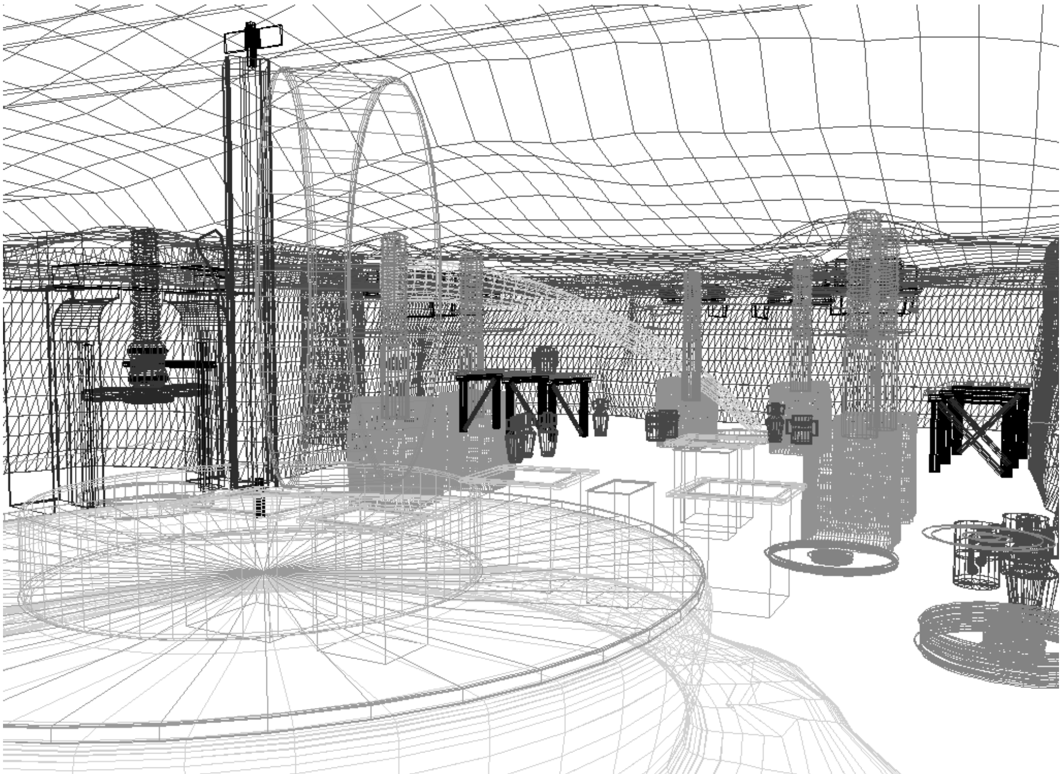

Figure 9

favoured the flow of the oil after the olives had been milled and pressed. The hypogeum can have a floor area from about 3.00 metres to 4.50 metres, thus obtaining an interior height of a minimum of 1.70 metres to about 3.00 metres.

Often the planimeter development seems to be rather articulated and definable as an “organic” structure; so the use of NURBS surfaces for the surveying of the environment has been highly productive both for the comprehension of the tri-dimensional development of the structure and for the geometric/constructive study in plan and section. Even the “machinery” for the treatment of the olives (milling-tanks and millstones) has been carefully reconstructed; this has brought out the planimeter relationships between the parts and their spatial impact inside the working space.

3. CONCLUSIONS

The profile of the curve-sections carried out by traditional methods (direct and instrumental) was seen to be valid for the “macroscopic” restitution of the hypogeums under investigation. On the tri-dimensional models reconstructed by means of NURBS surfaces it was possible to begin studies on the building techniques and the planimeter development of the two hypogeums. However, where the poor state of conservation did not yield a proper reading of the spaces, or the chance to appreciate the details, the work centered on a virtual re-evocation. This represents a way of popularizing, of immediate communicativeness and of particular importance for the transfer of scientific knowledge to the tourist/culture sector [8, 11]. Greater precision in the modelling, where it is necessary, can be obtained by the

Figure 10

use of a three dimensional laser scanner, which enables us to describe the measurements in the form of perfectly manageable "clouds of points", after an adequate transformation, as NURBS surfaces.

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ACCORDING TO THE DOCUMENTS IN ARCHIVES: THE DAMAGES AND THE RESTORATIONS IN DOLMABAHÇE PALACE- CEREMONIAL HALL AFTER THE EARTHQUAKE IN 1894

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ABSTRACT

Dolmabahçe Palace is a group of construction which reflects, in the second part of 19^{th} century, the new style in Ottoman architecture with its decorations and its synthesis.

During the reign of Sultan Abdülmecid, Dolmabahçe Palace with its annexes completed in 1856, is one of the monumental symbol of the innovations adopted from the west.

Ceremonial Hall (Muayede Salonu), which has an important place inside the palace has been seriously damaged during the Marmara earthquake in August 17, 1999. In this paper, both the damages in 10 July 1894 in Istanbul earthquake, the immediate restorations after, the material used, the artists worked have come to light with the documents from the archives. At the end of our paper we have found out that the damages on the dome, arches and vaults have been restored both with traditional materials and new technology during the both restorations.

1- INTRODUCTION

Dolmabahçe Palace is the main building among the groups which cover the administration of the State and the Sultan's House. This building, different from the traditional palaces which are usually seen as independent pavillions, has been arranged in a new style. As a result of new tendency, Mabeyn-i Hümayûn (Administration Building), Ceremonial Hall and Harem-i Hümayûn (Sultan's House) have come under the same roof functionally (Figure 1).

Ceremonial Hall connects two groups with its situation in the palace. It has been used for the ceremonies of Bairam, state functions and historical meetings since it has a special place with its architecture and with its inside and outside decorations (Figure 2).

This hall demonstrates the importance and magnificence of imperial administration: It covers an area of 2000 m2, surrounded by 56 columns over which rises a splendid dome to a height of 36 meters (Figure 3). The outside surface of the dome was completed with the lead covered wooden construction that can also be seen in all roofs of the palaces, suspended from this unique dome is an English chandelier weighing 4.5 tons [6,10].

The Ceremonial Hall which is the most grandiose place in the palace has been effected from the earthquakes occurred in 1894,1901 and 1999 [2].

The biggest earthquake of 19 th century in Istanbul occurred in July 10, 1894. The 1894 earthquake which was felt in a large area outside Istanbul caused a lot important damages in Istanbul and its surroundings [5,7].

Upon the wish of the Sultan Abdülhamid II, a report on this earthquake was prepared by D. Eginitis who was the Director of the Observatory in Athens. Sarkis Balyan, Berthier, R'aimondo d'Aranco and Aleksandre Vallaury were appointed as experts for the damaged buildings and their restorations.

2- THE DOCUMENTS IN ARCHIVES

From the correspondences of the era, some parts of Dolmabahçe Palace and the Ceremonial Hall had been damaged during the above earthquake.

The documents in the Ottoman Archives of the Prime Ministry [8] and the ones in the archives of Hazine-i Hassa National Palaces have been an important source on this subject. According to the documents in the Ottoman Archives of the Prime Ministry ; An expert report and some permission forms on the damaged parts of Dolmabahçe Palace- Ceremonial Hall were handed to Sultan Abdülhamid II by a group of engineers and the supervisor of workmen of Administration of the Sultan's Properties (Ebniye-i Seniyye İdaresi) through the Mikael ; The Minister in Charge of the Sultan's Properties (Hazine-i Hassa Nazırı).

We came across the following information of the documents of the estimation of the damages dated R. 5 th September 1310 (1894) (Figure 4-5).

It was stated that there had been a big scaffold both in and out of the Ceremonial Hall and in order to consolidate 6-7 centimetres crack on the main part of the upper levels of the eaves and materials such as iron, cement etc. had been used.

However, the lead which was damaged on the pendants and arches inside the dome had been painted in oil. The window at the side wall on the first floor was repaired in the original way. The other splited and broken parts, chasing, stucco and plaster ceiling decorations had also been restored. For these restorations total approximate price has been estimated as 162. 309 kuruş.

After the first of the documents which determined this restoration with its main lines, another one which completes and gives the detail is in the Archives of the National Palaces Department , Document Number II/ 506 are [9] :

1-Weekly vouchers of the repairing of Administration Building and its balustrades in Beşiktaş Palace (Dolmabahçe Palace) after the earthquake (Figure 6).

2-The tenth weeks' vouchers of repairing of railings of balconies of the Ceremonial Hall and the ones at the seaside of the Beşiktaş Palace (Figure 7).

3- The title of the archive document is about the voucher of the first week of the dome of the staircase and Storeroom in Beşiktaş Palace (Figure 8).

From the documents dated 1894-1895 we understood that there had been some restorations on the roof of the Ceremonial Hall in Dolmabahçe Palace and on the balustrades, floor, stairs going into the Ceremonial Hall, the staircase and the Imperial Storeroom.

In the documents, the place recorded as the Imperial Storeroom connected with the Ceremonial Hall could be the space called today as (000) Storeroom in the basement.

However a material list has been prepared to be used in the restorations in the Palace.

In this list dated 23 rd July ,1894 there has been two double cuts, large bobbins made of the wood of balsam tree and another two with single cuts and English rope of tar (Figure 9).

In the same group of documents there are also list of tallies. We learn the names of the workmen who were in the restoration, and their occupations from it..

In these reports dated 11 July 1894-22 July 1894 for the first time the restoration of the Ceremonial Hall and the balustrades facing to seaside damaged by the earthquake had been taken into account. For these restorations stonemakers, carpenters and workmen worked mostly.

Stonemakers of the first week are Bodos Yordan, Yerasimo Hristo, Yorgo Toma, Mustafa Hüseyin, Yuvan Yali, Gokas Agop, Pandeli Avram, Kupel Bedros, Bali Yordan. Vasili Lazari and Kidiyki Tail joined them as stone makers in the second week (Figure 10).

We understand that in the following fortnight journals, majority of the workmen were of stonemakers and carpenters, another group of twenty workmen points out for us that there was a restoration on the seaside facade of the palace. The existence of a group of carpenters reminds us the probability of door and window repairings and general carpentry.

During the first months of 1895 they had to work inside because of the weather conditions. During the month of 20 February 1895 - 6 March 1895 and 9 March 1895 - 11 March 1895 the roof of Ceremonial Hall, staircases, the Imperial Storeroom have been repaired. The group of professionals are in the following :

During the first week (7) plaster workmen, (6) carpenters, (2) mortar mixers, (1) tinman, (2) stone makers, (2) plumbers, (7) painter of ornaments, (2) sweepers.

At the second week (20) wall painters, (9) carpenters, (9) engravers , (2) lead melters,(1) glass maker.

According to the documents it was known that 145 workmen had worked in special places mentioned above.

This document has shown us that after finishing the rough work, the restorations in the ceiling decorations on the walls have been totally completed.

The name of the painters are:

During the first week: Master Kalenot, Master Rafael(Refail), Major Bogos, Manuel, İskender, Kirkor Baldasar (Figure 11).

During the second week: Master Koloska,Master Refail, Major Bogos,Manuel, Mardik Sergis , Eris (Iris), Minor Bogos, Kirkor Baldasar, Master Istepan, Master Kazbar,Master Kirkor, Bedros Melkon Karait, Enderyas Mesrop, Agop Kirgorik, Kazbar Agop, Kirkor Haçik, Hacı Ohan, Sergis Kirkor.

Head painter of ornament is Master İsmail.

These documents also give us an idea about the materials which we used in the Ceremonial Hall. It also gives us some knowledge about the work done and plaster work and paintings on the masonry wall. Some canary yellow, rosepan red, charcoal pencil drawings with sable brush show us the renewal of the wall paintings.

From the vouchers in the documents we understand that French plaster, minimum cement, lime marble, Grenoble cement, Hülyen cement, Malta stone have been bought. As wooden material pine of Sinop was used. After all these steps, floor parquet was polished (Figure 12). They are 1 kıyye (400 dirhem or 2.8 lb) [3]. of olive oil, half kıyye of cotton, 1 kıyye of pure resin were bought from a company called "Tatyos Moldovack and Sons " in Istanbul (Figure 13).

The same documents gives us an idea about the traditional materials used in polishing of the Ceremonial Hall.

4- THE EFFECT OF THE 1999 MARMARA EARTHQUAKE IN THE CEREMONIAL HALL AND RESTORATION ACTIVITIES

The greatest damage was seen in the Ceremonial Hall of Dolmabahçe Palace including some rooms and halls too.

It was known that the wooden roof of Ceremonial Hall was transferred into steal construction in the year 1950's [4] (Figure 14). The joints of this construction were broken too. There has been cracks and separations 5-8 centimetres wide on the alum- layers of the joining arches and on the main dome (Figure 15). With the cracking a great part of the plasters , one of the pendants falling down on the floor, a big crystal candle stick was broken and damaged the parquet floor (Figure 16).

On the galleries of the Ceremonial Hall, the domes of the rooms in the corners and on the walls there were serious cracks. There were openings on the balustrade, in the window frames and on the stone plates along eaves facing to the seaside. A large scaffold has been prepared to repair the damages. The alum cover which was a heavy burden on the domes and their joints have been removed and the cracks have been filled with Horasan mortar. While this was done a kind of traditional mortar which is a kind of binder puzzolan hydraulic mortar with mechanical strength was injected in order to consolidate.

The same procedure has been applied on the pendants and arches. Furthermore epoxy binders with carbon fibres have been applied on the counterbracing and the domes. After the work of consolidating and the basic job was done , the painting and gilding restorations were completed [1] (Figure 17).

The renewal of the roof construction of the Ceremonial Hall with the steel structural materials and the addition of the new reinforced concrete materials in the Early Republican Period , and their excess, severe burdens to the building should be analysed , too.

Constructional elements, traditional and up to date building materials must be controlled according to their physical, chemical and mechanical characteristics.

CONCLUSION

It has been found out that the damages in Dolmabahçe Palace-Ceremonial Hall, during 10 th July 1894 earthquake and the 17 th August 1999 earthquakes have been seen on the same parts and on the same constructional elements. In both restorations after the two earthquake traditional and recent materials have been used. During the 1894 earthquake Horasan mortar , some cement with European origin have been used for the restorations.

Today it has been consolidated with Horasan mortar besides puzzolonic hidrolic lime binder. At the same time carbon fibers with epoxy binders have also been used in these consolidations.

On the restored dome, vaults and arches the paintings have been completed with the traditional materials and methods.

In the above respect ;

a) In the conservation and the restoration of the historical buildings, the repairing of the architectural elements and the ornaments which have been realized during historical era, must have been closely studied.
b) The special materials and the methods used in the previous restorations must be supported by the documents in the archives.
c) The restoration activities and works ought to have been done by the evaluation of both traditional and modern building materials and methods.

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Figure 1
General view of Dolmabahçe Palace

Figure 2
Photo of Dolmabahçe Palace and Ceremonial Hall taken by B. Kargopoulo during the reign of Abdülhamid II. Library of University of Istanbul (IUK)

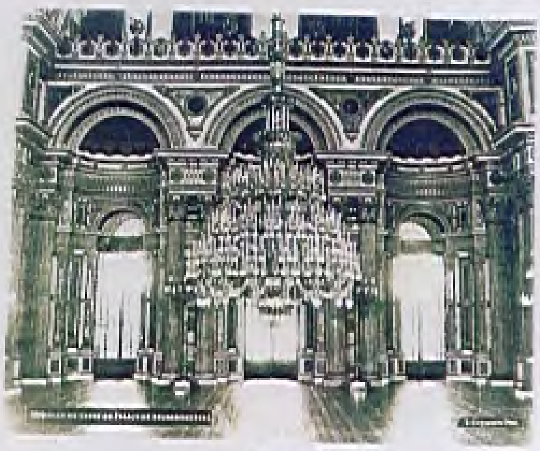

Figure 3
The inner view of Ceremonial Hall taken by B. Kargopoulo, during the reign of Abdülhamid II . Library of University of Istanbul (IUK).

Figure 4
The permission forms of damaged parts of Dolmabahçe Palace-Ceremonial Hall by the Imperial Treasury. Ottoman Archives of Prime Ministry (BOA), Y.Mtv.nr.105/21

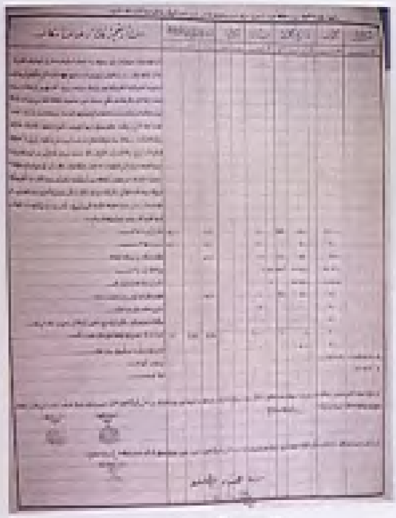

Figure 5
Billing for quantities for the damaged parts of the Imperial Treasury in Dolmabahçe Palace, BOA ,Y. Mtv. nr. 105/ 21

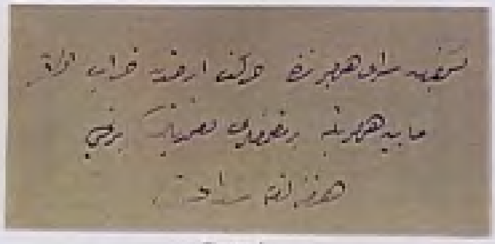

Figure 6
The repair of the balustrades in the Administration Building. The Archives of National Palace (MSA) Document number II/506-1

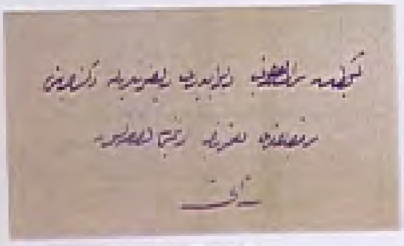

Figure 7
The repair of the railings balkonies in the Ceremonial Hall at the seaside of Dolmabahçe Palace. MSA, Document number II/506 -2.

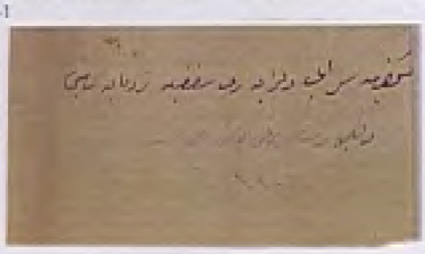

Figure 8
Repairing of staircase, Imperial Storeroom and the roof of the Ceremonial Hall. MSA, Document number II/506 -3.

Figure 9
The materials used for the scaffold inside the Ceremonial Hall . MSA, Document Number II/506-1e.

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Figure 11
The list of the people who worked at the roof of the Ceremonial Hall, The Imperial Storeroom MSA, Document number II/506-3t

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The voucher of the Grenoble cement and etc. MSA, Document number II/ 506-1c

Figure13
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Figure14
The wooden roof of Ceremonial Hall transferred into steel construction

Figure15
The cracks and seperations on the main dome and joining arches.

Figure16
Repairing of the Ceremonial Hall the chasing and plasters torn down from the pendants which was facing the seaside.

Figure 17
Photo of the Ceremonial Hall after the restoration.

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IDENTIFICATION AND DOCUMENTATION OF A BUILDING TYPE IN THE VILLAGE SETTLEMENTS OF THE OTTOMAN PERIOD: LAUNDRIES

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ABSTRACT

The architectural products are among the meaningful indicators of the cultural identities of societies. Together with monumental buildings, modest ones are the considerable evidences, which reflect the social structure and the life styles of the past settlements. As it is known, buildings display design characteristics and construction techniques at the junction of the need to meet various living and societal necessities as well as geographical, technological, economic and esthetical factors.

The aim of this study is the documentation of the design characteristics and construction techniques of the building type, which is not taken into account among the researches up to now. This type of building is the laundry, which serves the inhabitants of the settlement. The research is developed with the method consisting of the documentation of the field surveys, and the evaluation of the obtained data. The examples determined in the village settlements near Urla in the Western Anatolia are planned while relating to the fountain integrally. The laundries exhibit a certain scale relationship with the connected fountains. In these buildings, the installation of a water system is remarkable.

These buildings, which exhibit cultural traces and accumulation of the settlements of the Ottoman Period near Urla, have special importance since they display a developed social system. This research displays a dimension that determines the values and locations of these buildings, which are small in scale but great in societal terms.

1. INTRODUCTION

Buildings constructed in line with the needs of the society throughout history are multi-dimensional evidences of the past experiences. They are considered as physical data defining the stages of the social and technological developments of the societies and they form a memory connecting the past with today. The aim of this study is to introduce laundries in the village settlements in Anatolia. These buildings can be evaluated as an indicator of a societal level in the village life although they are out of use due to the changing and developing conditions. These buildings are historical values identifying the system of the social life of their periods and indicators of the Turkish identity at places they are built.

The study is carried out with the advice of Prof. Tuncer Baykara from the Department of History, Faculty of Letters of Ege University. One of the two laundries, which Prof. Baykara has identified and suggested me for studying, is in Denizli district of Urla and the other is in the village of Özbek. Denizli, which is a district today, was in the status of a village before 1981. During the study another laundry is identified in Denizli, and two more are determined in Özbek of which one is collapsed. Apart from these, another example is determined in the village of Kocadere that was also turned into a district of Urla in 1981*. Definite data could not be received in the registry archives to determine their exact construction dates. The date of 1862 is written on the fountain closer to one of the examples in the village of Özbek, which is clumsy rebuilt in Yenipınar district. It can be argued that, the laundries and related fountains that are studied in these villages, which are Ottoman settlements by the 16th century, can date back to earlier times from the 19^{th} century. For exact dating, our knowledge is not enough yet.

The examples determined in the village settlements around Urla are examined in between June and July 2000. In the context of this study the planning and construction characteristics of these buildings are introduced and evaluated.

2. GENERAL LOCATION AND HISTORICAL EXAMINATION

Urla is to the west of İzmir and in the centre of the peninsula with the same name. On the west side of the horn stretching towards the north the village of Özbek, and on the east side Denizli and Kocadere are located. Each of the settlement is 9km away from the centre of Urla. The region is settled starting with pre-historical eras and was conquered by Turks under the Aydınoğulları hegemony in the 14^{th} century [1]. As Urla is situated on an important trade route from Çeşme to the inner parts of Anatolia, from those times in Urla and the settlements around it

* Denizli and Kocadere were made a district of Urla on 21 September 1981 in order to provide them with sufficient municipality services with an order by Ege Ordu ve Sıkıyönetim Komutanlığı (Commandership of Aegean Army and Martial Law). Municipality of Urla, File Archives.

have a dense Turkish population [**][2]. In the 16th century, Hafsa Hatun, mother of Kanuni Sultan Süleyman devoted the revenues of Urla and its surroundings, which was a developed region economically, to the buildings in Manisa [4]. 16th century foundation charters show this event [3]. The region continued to prosper during the Ottoman Period and in the 18th and 19th centuries Greeks migrated. The reason for this was the lack of workers in the region [7]. However, it is understood that the villages where the research of the buildings is done, Denizli, Kocadere and Özbek are Turkish villages in the 19th century [8].

3. EXAMINATION OF THE BUILDINGS

The field studies on the buildings are based on measured survey and photographic documentation. In order not to exceed the limits of the paper all documents could not be presented here.

3.1. The Laundry closer to the Mosque in the Village of Denizli

The mosque in the village of Denizli is located on a rocky spot closer to the beginning of the forest road on the edge of the stream. The laundry is located on the southeast of the mosque and adjacent to a fountain and together with the mosque defines a square (Figure 1). This square, which has an old plane tree, is perceived as a remarkable exterior space in the settlement. The mosque, laundry and square are separated from the woods which stretches through the south and the west by the stream over which an old bridge exists.

The date of construction and the donor of the mosque could not be determined. The building consists of a single space and square in plan. The superstructure of it is a dome and covered with tiles from the exterior. The minaret is located on the northwest corner. South and west walls are supported with buttresses. The north, east and partially south and west of the building is surrounded by boundary walls and the space between the walls and the building is used as a cemetery. Two cypress trees and the tombstones are historical values to be preserved.

The laundry, situated at the southeast of the mosque, is constructed in relationship with the fountain on its northwest (Figure 2). It measures internally 4.70m. at its north and south and 3.72m. at its east and west sides and is composed of a single rectangular space (Figure 3). The superstructure of the building is completely collapsed today. However, it is understood from the only one timber beam and the condition of the walls that the building was originally covered with

[**] It is understood from the statistical information belonging to the XVth and XVIth centuries that the majority of the population is Turkish. T. Baykara, 1980. "XIX. Yüzyılda Urla Yarımadasında Nüfus Hareketleri", Türkiye'nin Sosyal ve Ekonomi Tarihi (1071-1920), Birinci Uluslararası Türkiye'nin Sosyal ve Ekonomik Tarihi Kongresi Bildirileri, Ankara, pp.279-286.

a gabled roof. The entrance is on the east side facing the square where the plane tree is situated. West wall is 1.20m. away from the main rock to the east. The entrance to this space between the building and the main rock is provided through a door opening arranged on the middle of the west wall. The northern part of this open rectangular space is the narrow surface of the fountain. In this part, water is taken through a terracotta water pipe from the fountain to the basin, which is arranged at the ground level. Then water is taken to another basin in the laundry again with a terracotta water pipe. A channel starting at the basin in front of the west wall in the laundry continues throughout the floor and carries the water to outside from the entrance.

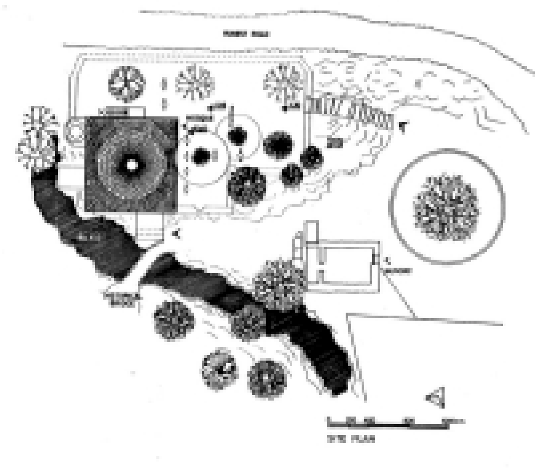

Figure 1. Site plan

Figure 2. General view of the laundry

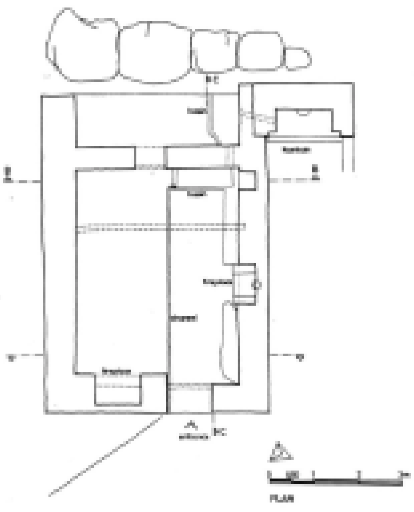

Figure 3. Plan

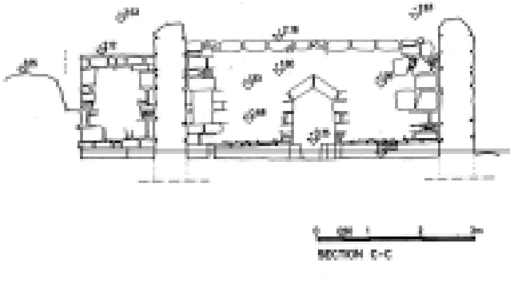

Figure 4. Section C-C

Inside the laundry, a depressed arched fireplace is arranged on the east wall to the south of the entrance opening. On the north wall, almost at the centre of the wall a triangular arched fireplace and a niche at the western part of the same wall are arranged (Figure 4). The niches are determined in the other examples also and according to the elderly of villages were used to put oil lamps or candles. Stone platforms ("seki"s) are arranged at the both sides of the fireplace in front of the north wall. As there are the ruins of the roof in front of the south wall it could not be determined the elements of this wall.

The walls are constructed from rubble stone. Cut stone is used on both sides of the entrance opening, partly on the corners of the north wall and the voussoirs of the fireplaces. On the right part of the entrance opening a re-used stone is determined on which cavities of clamp are seen. Today the whole superstructure of the building is collapsed however it can be understood that it was a timber roof.

The fountain is 1.20m. x 2.65m. and 1.40m. at height. Front face is limited with projected parts on both sides. It is in the form of a niche and is spanned by a depressed pointed arch. The waterspout, which is in the form of a lion head, is in a very deteriorated condition.

When the junction part of the laundry and the fountain is examined, it can be understood that the fountain was built before and the laundry was constructed later adjacent to it. The fountain is in use today and its water is drinkable. However, since the superstructure of the laundry is collapsed the building is in a state of ruin.

3.2. The Laundry on Denizli Street

The second laundry building in the village of Denizli is also constructed adjacent to a fountain (Figure 5). This second laundry is in the northern part of the road, which is approximately 4 m. in width, called Denizli Street, reaching the square mentioned above where the mosque and the laundry are situated. The laundry internally measures 3.80m. x 3.82m. and it is almost square in plan. The entrance to the building is on the road side and hidden ingeniously by a wall constructed from rubble stone. On the south corner it is connected to the back wall and one side wall of the fountain. For this reason, in the square plan, the fountain makes a projection on the south corner. The superstructure, which was a single pitched timber roof, is completely destroyed. The entrance is on the southeast side, symmetrical part to the fountain. On the northwest wall that is across the entrance, a triangular arched fireplace is arranged. There is a platform, which is 0.60 m. high on the left side of it along the wall. On the southeast wall where the entrance is located there is bigger fireplace arranged as a rectangular recess. At the south corner, where the fountain makes a projection into the laundry, the original water connection detail can be seen at the southeast corner formed by the intersection of the walls belonging to the fountain and the laundry. However, later on, the water connection is taken to the other corner of the narrow wall of the fountain. Today, water is taken from the basin of the fountain to the buildings that are located on upper levels by a water pipe from this changed water entry point through the laundry by taking it above the wall height. It is used for irrigation of gardens. In front of the southwest wall next to where the fountain makes a projection a platform, which is 0.20 m. high, is arranged. On the northeast wall a triangular arched fireplace rectangular in plan and another smaller niche rectangular in plan are arranged. Afterwards, a concrete plate is placed in the first one. There is also another lower platform, which is 0.20 m high in front of this wall.

The walls are made of rubble stone. Although the superstructure is demolished completely the remnants of materials indicate to a timber roof covered with tiles.

When the wall connection parts are examined, in this example it can also be easily understood that it is seen that the fountain is constructed before than the laundry and the latter was constructed as to be adjacent with the former. Similar to the first case it is due to fast deterioration since the superstructure is destroyed.

Figure 5. The laundry on Denizli Street

Figure 6. The laundry in District of Terslik, Özbek

3.3. The Laundry in the Village of Özbek, District of Terslik

The laundry is on the road that passes through the Terslik district and stretches to Yenipınar and Kermen districts. It has a rectangular plan extending on the north-south direction and covered with depressed-pointed barrel vault. It is constructed 30cm away from the fountain to its northeast. The entrance to the building is from the south and arranged that it would not be seen from the road (Figure 6). The open space between the laundry and the next plot provides an entrance space and hides the main entrance from the road.

It measures internally 3.83m. at its north and south and 2.18m. at its east and west sides and is composed of a single rectangular space (Figure 7). The entrance is through depressed arched opening. Inside the laundry, on the east wall there are two depressed arched fireplaces (Figure 8), while on the west wall, two niches in almost square in plan, and on the north corner there is a small window opening. On these two long walls projected stones are left at certain distances from 1.55 height from the ground. It is learnt from the elderly that these stones were used to hang the laundry. On the northern wall, which is across the entrance, opening there lies the terracotta water pipe. Since the floor of the building is filled with earth the basin that should be below the water pipe could not be seen.

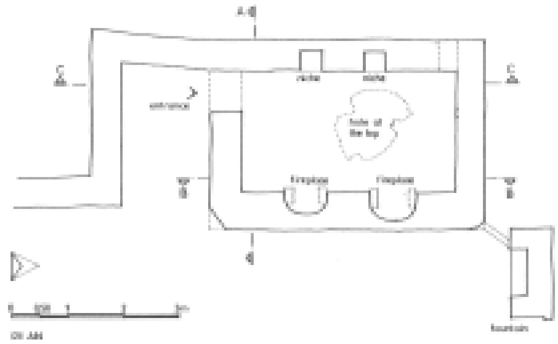

Figure 7. Plan of the laundry in District of Terslik, Özbek

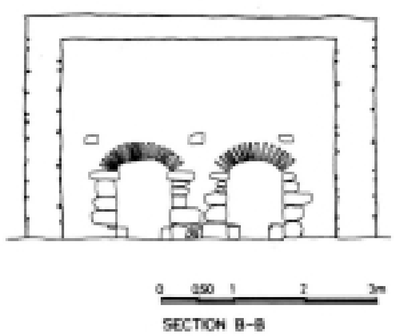

Figure 8. Section B-B

The walls are made of rubble stone and the superstructure, which is vault, is made of brick. Brick is also used on the arched parts of the entrance opening and the fireplaces. In the middle part of the vault bricks are placed in the east-west direction. On the sides they are bonded as to form the curvilinear parts on the north south direction. A part the vault is collapsed almost at the centre of the building. The dimensions of the bricks are measured as 24-25 x 17-18 x 3.5cm.

The dimensions of the rectangular planned bricks are different from the square planned early Ottoman period bricks which are 27.5-29.5 x 3.5-4.5 cm. in dimensions [6] and the bricks which are 24 x 35 x 3.5 cm. in dimensions used in the 17th century Ottoman buildings [5]. However, they are half of the brick sizes of the 17th century dimensions.

The dimensions of the fountain at the northeast corner of the laundry are 0.75m. x 1.67m. As the ground level is raised the basin remained below the ground. Its height is 2.00m. from the present ground level. The front face is limited with projected parts on both sides. Depressed pointed arch, which forms the fountain niche, is made of cut stone stands on these parts. Since the ground level is raised by earth waterspout is almost at the same level with the ground. The triangular formed niche that is a little above and on the right side of the waterspout is said to be a place for candle by the villagers.

As in the other examples, it is thought that, the laundry is constructed after the fountain. Since the fountain does not supply water today, both structures are not in use and they are under the risk of disappearing.

3.4. The Laundry in the Village of Özbek District of Yenipınar

This laundry is located at a closer place to the stream on which there is a historical bridge and it is totally rebuilt (Figure 9). As the walls, fireplaces and the super structure of the building is reconstructed by using brick and concrete, the original materials could not be determined. The villagers said that it was in use until 20 years ago. The inscription that is at the entrance opening dating to 1938 is definitely of a repair period. It measures internally 5.10 m. at its north and south and 3.72 m. at its east and west sides and is composed of a single rectangular space. Inside the building the trace of a channel starting at the basin, where the water coming from the fountain is held, can be determined on the floor. The fountain, which is in front of the entrance part of the laundry, keeps its original features to a large degree. The fountain is made of cut stone and whitewashed afterwards, the inscription is dated to 1248 H. / 1832 A.D. The west side facing the road measures 2.80 m. the south side facing to stream measures 1.96 m. and the narrow sides measure approximately 0.67 m. It is 2.62 m. high and arranged in the form of a partial polygon (Figure 10). The angle between the long sides is approximately 110^0. The parts facing the road and the stream are in the form of pointed arched niche. The basin parts are now below the ground level and the fountain does not supply water today. It can be determined that water is taken to the laundry from the basin at the south facade facing the stream.

Figure 9. The Laundry in District of Yenipınar, Özbek

Figure 10. The fountain in District of Yenipınar, Özbek

3.5. The Laundry in the Village of Özbek District of Kermen

The laundry does not exist today. The remains of its stones can be determined on the right side of the fountain and a new building is constructed on its plot. However, the fountain to which it was adjacent is still intact. The dimensions of the fountain are 1.10m. x 2.10m. and it is 2.05 high. It is made of rubble stone. The front part is in the form of depressed pointed arched niche. The water pipes carrying the water to the laundry can be determined by the traces on it. Since the fountain does not supply water today it is not in use.

3.6. The Laundry in Kocadere District

The laundry that is at the southwest of the Kocadere Mosque is thought to be belonging to a later period than the ones examined in the villages of Denizli and Özbek. The building is located on the west side of the main road of the village and above 1.20m higher from the road level. The old fountain that was adjacent to the south wall was destroyed 10 years ago according to the information the villagers gave. Instead of it the clumsy new one is constructed from reinforced concrete. The laundry has a rectangular plan extending on the north-south direction. The superstructure is a gabled roof. It measures internally 4.52m. at its east and west and 2.90m. at its north and east sides .The entrance to the laundry is provided from the west making it sure that it is not seen from the road. On the west wall where the entrance is provided, there are depressed arched two fireplaces and between them at the level of the arches a rectangular niche is arranged. On the east wall there are two small window openings and between them a niche is arranged as well. On the ground level of the south wall where only a niche exists, the part providing the connection with the fountain is closed by concrete.

The building is made of rubble stone, and the arches of the fireplaces are of brick. The sizes of the bricks used are 20 x 10 x 5-6 cm. and this shows that the building is from a later period.

4. EVALUATION

The laundry spaces are designed and provided the possibility of common use in the apartment blocks in foreign countries but are not common in Turkey. This study documents that these spaces are designed as an independent building in the village settlements during the Ottoman Period and exemplify an aspect of the life style of the Ottoman society. In addition, the study clarifies a certain phase of the way of life in the Ottoman society, which inhabited in the village settlements. Generally, it is known that the laundry is done along the streamside in the villages. However, these documented laundries display the fact that this action was done in a building, which is an indicator of the societal level of the period.

The examined laundry buildings are type of social building that was in each district in the village settlements. These buildings that were put for the common use of the district were also used as small baths. At the time when they were constructed water installation was not available in houses. The importance they had was clear as they were used to meet the need of laundry and bathing. According to the information received from the elderly of the settlements the population living there used the buildings in an order. They used the building with their neighbouring relationships in an order lacking any interference of an official authority or organisation. During the daytime, women did the laundry and had their baths with their kids, and at night men used the place for bathing purposes.

The buildings are made of rubble stone as a modest solid mass and were not plastered, they exhibit an introvert design. At the entrance openings wings were not used. In some examples, entrance openings were hidden from the road through planning, and in all of the examples a curtain was hanged so that the relationship with the outside is prevented. Lighting is provided by oil lamps and candles. The interior is designed with the minimum constructional elements that the function required. It is a significant feature that, water is taken into the building either from a closer fountain or an adjacent one, and then used water is also taken away.

5. CONCLUSION

The construction dates of these buildings that we studied around Urla could not be determined. However, as it is stated before these villages have been Turkish settlements since the 16^{th} century. When the construction techniques, materials and characteristics are examined it can be argued that they are constructed either in the 18^{th} or 19^{th} centuries. These are the type of buildings with historical importance and demonstrate the traces of the Ottoman period around Urla, and they carry a great social function in our cultural inheritance. These small-scale buildings reflect the level of the social life of the society that they belong to. For this reason, these buildings, which can be seen as small architectural inheritance, are to be evaluated very carefully together with the monumental historical buildings as they show the cultural and social identity of the society.

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THE REMAINING RATE OF JAPANESE MODERN ARCHITECTURES IN TOKYO AFTER 20 YEARS

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ABSTRACT

The purpose of this paper is to observe and analyze the status quo of Japanese Modern Architectures in the 23 wards of Tokyo.
The word "Japanese Modern Architecture" means the architecture which was built in the period from Meiji era to the end of World War II in Showa era (1868 1945).
Architectural Institute of Japan researched Japanese Modern Architectures and made a list of them in 1980. About 13,000 buildings were listed all over the Japan. In the 23 wards of Tokyo, 2191 buildings were listed.
The research of Japanese Modern Architectures in The 23 wards of Tokyo had done after 20 years from the research of Architectural Institute of Japan in 1980.
Result shows that 1) less then 40% of Japanese Modern Architectures remained since 1980, 2) more than 60% of them remained in universities etc.

1. INTRODUCTION

PURPOSE AND BACKGROUND OF THIS INVESTIGATION

In late 80s, Japanese economy was in its active period and we called it "Bubble Economy".
In that period, many reconstruction plans were performed and many old architectures were replaced by big and new office buildings.
This investigation aims to research and grasp the situations of locations of buildings within Tokyo 23 wards listed in "The Pandect of Japanese Modern architectures" which was published by Architectural Institute of Japan in 1980. In this paper, we grasp the remaining and disappearing situation as a whole and their rates in each constructed era.

2. JAPANESE MODERN ARCHTECTURE

In Japanese architectural history, Japan terminated the feudal era (Edo era) in 1868 and started importing Western civilization from Meiji era. As the consequence of it, buildings affected by Western civilization were constructed.
Then the era changed into Taisho, then to Showa. The next big historical mark was the end of World War II. The Western architectures that were constructed between Meiji era and the end of World War II (1945) are called"Modern

Architecture"

3. SUMMARY OF THIS INVESTIGATION

The investigation was done from June 1998 till the end of May 1999 with filling the investigation form. The column items of the form are building name, constructed year, number of floors, etc. based on the contents of the pandect. The sums are made in each ward.

4. THE REMAINING AND DISAPPEARING SITUATION AS A WHOLE.

The total number of modern architectures listed in the pandect is 2,191. The number of remaining is 830 (37.76%) and the number of disappearing is 1,360 (62.20%). Over 60% of the modern architectures had already disappeared (see Table-1). There are 17 wards which still keep 25 buildings or over and the rest of this investigation focuses on these 17 wards hereafter.

(1) Remaining rate
Just 2 wards exceed 50% in the remaining rate. The highest rate ward is Toshima-ward (57.45%), then Meguro-ward, Suginami-ward, Kita-ward in the order of the rate.

(2) Disappearing rate
3 wards exceed 70% in the disappearing rate. The highest rate ward is of Chuo-ward (75.09%), then Nakano-ward, Sumida-ward, Ohta-ward in the order of the rate.

(3) Remaining and disappearing situation in University campuses.
14 wards have universities and the number of all modern architectures in these campuses is 229 in the pandect. The number of remaining architectures among them is 148 (64.63%) and the number of disappeared architectures is 81 (35.37%). The remaining rate other than campuses is 34.96% and we can see that the remaining rate in campuses heightens the total rate of remaining architectures.

5. REMAINING SITUATION IN EACH JAPANESE ERA.

In the pandect, the number of modern architectures built in Edo era(1603 – 1868) is 1 (0.05%), in Meiji era (1868 – 1901) is 86 (3.93%), in Taisho era (1911 – 1925) is 451 (20.58%), and in before-the-war Showa era (1925 – 1945) is 1327 (60.57%). The Showa-era architectures count over the half of all remaining ones.

The highest remaining rate is in Meiji era, then Taisho, Showa in the order of the rate and we can see the tendency that the older the architecture is, the higher the remaining rate is. The remaining rate of Meiji-era architectures is

55.81% (see Table-2).

The wards which keep the highest number of Meiji era architectures are Minato and Bunkyo-ward, and each ward keeps 10 architectures. The ward that keeps the highest number of Taisho era architectures is Bunkyo-ward, then Chiyoda-ward, Minato-ward, Toshima-ward in the order of the number. The Chuo-ward lost a lot of architectures but the other wards still keep 20 architectures and over.

Table-1 : The remaining and disappearing situation as a whole

Ward		Total remaining situation in each ward				Remaining situation in campuses			
		whole	remaining	disappearing	unknown	whole	remaining	disappearing	unknown
Chiyoda		307	102	205	0	9	3	6	0
	%	100	33.22	66.78	0	100	33.33	66.67	0
Chuo		265	65	199	1	0	0	0	0
	%	100	24.53	75.09	0.38	0	0	0	0
Minato		237	75	162	0	16	11	5	0
	%	100	31.65	68.35	0	100	68.75	31.25	0
Shinjuku		127	59	68	0	30	21	9	0
	%	100	46.09	53.91	0	100	70.00	30.00	0
Bunkyo		223	107	116	0	60	43	17	0
	%	100	48.21	51.79	0	100	71.67	28.33	0
Taitoh		146	63	83	0	9	8	1	0
	%	100	43.15	56.85	0	100	88.89	11.11	0
Sumida		39	11	28	0	0	0	0	0
	%	100	28.21	71.79	0	0	0	0	0
Kohtoh		34	14	20	0	4	4	0	0
	%	100	41.18	58.82	0	100	100.00	0.00	0
Shinagawa		78	31	47	0	2	1	1	0
	%	100	39.74	60.26	0	100	50.00	50.00	0
Meguro		69	36	33	0	25	14	11	0
	%	100	52.17	47.83	0	100	56.00	44.00	0
Ohta		90	27	63	0	23	7	16	0
	%	100	31.46	68.54	0	100	30.43	69.57	0
Setagaya		90	34	56	0	3	1	2	0
	%	100	37.36	62.64	0	100	33.33	66.67	0
Shibuya		143	45	98	0	12	8	4	0
	%	100	31.47	68.53	0	100	66.67	33.33	0
Nakano		40	10	30	0	0	0	0	0
	%	100	25.00	75.00	0	0	0	0	0
Suginami		124	61	63	0	9	7	2	0
	%	100	49.19	50.81	0	100	77.78	22.22	0
Toshima		47	27	20	0	25	18	7	0
	%	100	57.45	42.55	0	100	72.00	28.00	0
Kita		53	26	27	0	0	0	0	0
	%	100	49.06	50.94	0	0	0	0	0
Arakawa		20	7	13	0	0	0	0	0
	%	100	35.00	65.00	0	0	0	0	0
Itabashi		14	6	8	0	0	0	0	0
	%	100	42.86	57.14	0	0	0	0	0
Nerima		19	11	8	0	2	2	0	0
	%	100	57.89	43.11	0	100	100	0	0
Adachi		17	5	12	0	0	0	0	0
	%	100	29.41	70.59	0	0	0	0	0
Katsushika		7	4	3	0	0	0	0	0
	%	100	57.14	42.86	0	0	0	0	0
Edogawa		2	0	2	0	0	0	0	0
	%	100	0	100	0	0	0	0	0
Total		2,191	830	1360	1	229	148	81	0
	%	100	37.76	62.20	0.05	100	64.63	35.37	0.00

Table-2 : Remaining and disappearing situation in each ear

Ward		Edo			Meiji				Taisho			Showa			Unknown		
	Total	sub-t	rem.	disap.	sub-t	rem.	disap.	unkown	sub-t	rem.	disap.	sub-t	rem.	disap.	sub-t	rem.	disap.
Chiyoda	307	0	0	0	8	5	3	0	59	22	37	231	72	159	9	1	8
	100	0	0	0	100	62.50	37.50	0	100	37.29	62.71	100	31.17	68.83	100	11.11	88.89
Chuo	265	0	0	0	3	1	2	0	49	9	40	195	57	138	18	4	14
	100	0	0	0	100	33.33	66.67	0	100	18.37	81.63	100	29.23	70.77	100	22.22	77.78
Minato	237	0	0	0	15	10	5	0	51	22	29	129	35	94	42	10	32
	100	0	0	0	100	66.67	33.33	0	100	43.14	56.86	100	27.13	72.87	100	23.81	76.19
Shinjuku	127	0	0	0	4	3	1	0	28	13	15	80	39	41	15	4	11
	100	0	0	0	100	75.00	25.00	0	100	46.43	53.57	100	48.75	51.25	100	26.67	73.33
Bunkyo	223	0	0	0	20	10	10	0	57	26	31	103	56	47	43	15	28
	100	0	0	0	100	50.00	50.00	0	100	45.61	54.39	100	54.37	45.63	100	34.88	65.12
Taito	146	0	0	0	8	4	4	0	21	7	14	107	47	60	10	6	4
	100	0	0	0	100	50.00	50.00	0	100	33.33	66.67	100	43.93	56.07	100	60.00	40.00
Sumida	39	0	0	0	1	0	1	0	8	2	6	22	8	14	8	2	6
	100	0	0	0	100	0	100	0	100	25.00	75.00	100	36.36	63.64	100	25.00	75.00
Kohtoh	34	0	0	0	3	3	0	0	6	3	3	21	8	13	4	0	4
	100	0	0	0	100	100	0	0	100	50.00	50.00	100	38.10	61.90	100	0	100
Shinagawa	78	0	0	0	1	1	0	0	14	5	9	31	15	16	32	11	21
	100	0	0	0	100	100	0	0	100	35.71	64.29	100	48.39	51.61	100	34.38	65.63
Meguro	69	0	0	0	0	0	0	0	4	0	4	56	32	24	9	4	5
	100	0	0	0	0	0	0	0	100	0	100	100	57.14	42.86	100	44.44	55.56
Ohta	90	0	0	0	1	1	0	0	22	8	14	52	14	38	15	4	11
	100	0	0	0	100	100	0	0	100	36.36	63.64	100	26.92	73.08	100	26.67	73.33
Setagaya	90	0	0	0	2	2	0	0	13	8	5	53	19	34	22	5	17
	100	0	0	0	100	100	0	0	100	61.54	38.46	100	35.85	64.15	100	22.73	77.27
Shibuya	143	0	0	0	4	1	3	0	35	11	24	67	20	47	37	13	24
	100	0	0	0	100	25.00	75.00	0	100	31.43	68.57	100	29.85	70.15	100	35.14	64.86
Nakano	40	0	0	0	1	0	1	0	8	5	3	18	3	15	13	3	10
	100	0	0	0	100	0	100	0	100	62.50	37.50	100	16.67	83.33	100	23.08	76.92
Suginami	124	0	0	0	3	1	2	0	28	15	13	75	33	42	18	9	9
	100	0	0	0	100	33.33	66.67	0	100	53.57	46.43	100	44.00	66.00	100	50.00	50.00
Toshima	47	0	0	0	4	2	1	1	19	16	3	22	9	13	2	0	2
	100	0	0	0	100	50.00	25.00	25.00	100	84.21	15.79	100	40.91	59.19	100	0	100
Kita	53	0	0	0	4	2	2	0	14	7	7	23	12	11	12	5	7
	100	0	0	0	100	50.00	50.00	0	100	50.00	50.00	100	52.17	47.83	100	41.67	58.33
Arakawa	20	0	0	0	1	0	1	0	2	1	1	8	5	3	9	1	8
	100	0	0	0	100	0	100	0	100	50.00	50.00	100	62.50	37.50	100	11.11	88.89
Itabashi	14	0	0	0	2	1	1	0	1	0	1	9	4	5	2	1	1
	100	0	0	0	100	50.00	50.00	0	100	0	100	100	44.44	55.56	100	50.00	50.00
Nerima	19	0	0	0	0	0	0	0	7	4	3	11	6	5	1	0	1
	100	0	0	0	0	0	0	0	100	57.14	42.86	100	54.55	45.45	100	0	100
Adachi	17	1	1	0	1	1	0	0	1	1	0	10	2	8	4	1	3
	100	100	100	0	100	100	0	0	100	100	0	100	20.00	80.00	100	25.00	75.00
Katsushika	7	0	0	0	0	0	0	0	3	3	0	3	1	2	1	0	1
	100	0	0	0	0	0	0	0	100	100	0	100	33.33	66.67	100	0	100
Edogawa	2	0	0	0	0	0	0	0	1	0	1	1	0	1	0	0	0
	100	0	0	0	0	0	0	0	100	0	100	100	0	100	0	0	0
Total	2,191	1	1	0	86	48	37	1	451	188	263	1327	497	830	326	99	227
	100	100	100	0	100	55.81	43.02	1.16	100	41.69	58.31	100	37.45	62.55	100	30.37	69.63

Note: In table-1, the numbers of architectures in Meguro-and Ohta-ward in the pandect are corrected.

6. THE REMAINING AND DISAPPEARING SITUATIONS IN EACH TYPE OF STRUCTURES.

(1) COMPOSITION OF EACH STRUCTURE TYPE IN THE PANDECT

Observing the whole architectures in the 23 wards listed in the pandect, we can see that the reinforced concrete structures (RC structures) count the biggest as 873 (39.84%). Then, the wooden structures come next as 819 (37.38%) and the both types cover almost 80% of all. The steel framed structures (S structures) are the least in number as 28 (1.28%) (see Table-3).

RC structures and wooden structures are almost the same in their numbers, but the distribution of the number of each structure type is different among wards. However, in generally speaking, the central wards like Chiyoda, Chuo and Bunkyo have more RC structures than wooden structures and the circumjacent wards like Ohta, Setagaya, Shibuya and Suginami have the more wooden structures than RC structures. Minato-ward has almost the same big numbers of both RC structures and wooden structures.

The fire-resistant architectures including RC structures, steel framed reinforced concrete structures (SRC structures) and steel framed structures (S structures) other than bricken structures count 1008. This number covers the 52.60% of all other than unknown types and exceeds the half of all.

(2) THE REMAINING AND DISAPPEARING SITUATION OF ALL 23 WARDS.

Observing the remaining situation of all 23 wards in each structure type, we can see that the highest number is of the bricken structures and its rate of remaining is 55.77%. Then SRC structures, S structures and RC structures come in order of the rate. The highest rate of disappearing is wooden structure and its rate is 63.05% (see Table-4).

From here, we want to see the features of remaining and disappearing situations of wooden and RC structures.

A. REMAINING AND DISAPPEARING SITUATION OF WOODEN STRUCTURES.

The wards that have 25 wooden structure architectures or over in the list count 11 wards. The ward that has the highest rate of remaining is Shinagawa-ward and the rate is 50.00%. Then Meguro and Kita come in order of the rate. The ward that has the highest rate of disappearing is Chiyoda-ward and the rate is 81.25%. Then Shibuya, Setagaya and Ohta come in the order of the rate. The wards which have a big number of architectures listed in the list are Minato, Shibuya and Bunkyo in the order of the number, but the disappearing rate is 70.71% high in Shibuya-ward.

B. REMAINING AND DISAPPEARING SITUATION OF RC STRUCTURES.

The wards that have 25 RC-structure architectures or over in the list count 9 wards. The ward that has the highest rate of remaining among them is Bunkyo-ward and the rate is 57.55%. Then Taito and Shibuya come in order of the rate. The ward that has the highest rate of disappearing is Chiyoda-ward and the rate is 72.55%. Then Chuo and Minato come in the order of the rate. The wards that have a big number of RC-structure architectures listed in the list are Chiyoda, Chuo and Bunkyo in the order of the number, but the Chiyoda and Chuo have the high rates of disappearing and Bunkyo has a high rate of remaining.

7. CONCLUSION

Leave a matter as it is, Tokyo will have few Modern Architectures. So, it is very important to preserve the precious modern architectures remaining in Tokyo.

Table-3 : Composition in each structure type

Ward	Total	wooden	bricken	S-struc.	RC-struc.	SRC-sturc.	other	unknown
Chiyoda	307 100	31 10.1	8 2.61	0 0	152 49.51	51 16.61	6 1.95	59 19.22
Chuo	265 100	15 5.66	2 0.75	3 1.13	144 54.34	19 7.17	5 1.89	77 29.06
Minato	237 100	100 42.19	10 4.22	1 0.42	84 35.44	13 5.49	4 1.69	25 10.55
Shinjuku	127 100	49 38.59	3 2.36	0 0	71 55.91	2 1.57	2 1.57	0 0
Bunkyo	223 100	97 43.5	4 1.79	0 0	106 47.53	4 1.79	3 1.35	9 4.04
Taitoh	146 100	22 15.07	3 2.05	2 1.37	94 64.38	4 2.74	4 2.74	17 11.65
Sumida	39 100	2 5.13	0 0	0 0	25 64.1	2 5.13	0 0	10 25.64
Kohtoh	34 100	1 2.94	2 5.88	0 0	28 82.36	1 2.94	2 5.88	0 0
Shinagawa	78 100	36 46.15	2 2.56	1 1.28	10 12.82	1 1.28	1 1.28	27 34.62
Meguro	69 100	35 50.72	0 0	2 2.90	19 27.54	7 10.14	1 1.45	5 7.25
Ohta	90 100	61 67.78	0 0	13 14.44	14 15.56	0 0	1 1.11	1 1.11
Setagaya	90 100	71 78.89	1 1.11	0 0	7 7.78	0 0	1 1.11	10 11.11
Shibuya	143 100	99 69.23	0 0	0 0	31 21.68	1 0.70	0 0	12 8.39
Nakano	40 100	21 52.5	1 2.5	1 2.5	3 7.5	0 0	0 0	14 35
Suginami	124 100	87 70.16	0 0	5 4.03	22 17.74	1 0.81	2 1.61	7 5.65
Toshima	47 100	21 44.68	8 17.02	0 0	17 63.17	1 2.13	0 0	0 0
Kita	53 100	32 60.38	1 1.89	0 0	16 30.18	0 0	3 5.66	1 1.89
Arakawa	20 100	5 25	3 15	0 0	12 60	0 0	0 0	0 0
Itabashi	14 100	2 14.29	3 21.43	0 0	8 57.14	0 0	1 7.14	0 0
Nerima	19 100	14 73.68	0 0	0 0	5 26.32	0 0	0 0	0 0
Adachi	17 100	13 76.47	1 5.88	0 0	3 17.65	0 0	0 0	0 0
Katsushika	7 100	4 57.14	1 14.29	0 0	2 28.57	0 0	0 0	0 0
Edogawa	2 100	1 50	0 0	0 0	0 0	0 0	0 0	1 50
Total	2.191 100	819 37.38	53 2.42	28 1.28	873 39.84	107 4.88	36 1.64	275 12.55

Table-4 (Left-half) : Remaining and disappearing situation in each structure

Ward	Total	wooden				bricken			S-structure			RC-structure		
		sub-t	remain	disap.	Unknw	sub-t	remain	disap.	sub-t	remain	disap.	sub-t	remain	disap.
Chiyoda	307	32	6	26	0	7	5	2	3	1	2	153	42	111
	100	100	18.75	81.25	0	100	71.43	28.57	100	33.33	66.67	100	27.45	72.55
Chuoh	265	15	3	12	0	2	1	1	3	3	0	144	45	99
	100	100	20	80	0	100	50.00	50.00	100	100	0	100	31.25	68.75
Minato	237	100	35	65	0	10	4	6	1	0	1	84	29	55
	100	100	35.00	65.00	0	100	40.00	60.00	100	0	100	100	34.52	65.48
Shinjuku	127	49	20	29	0	3	3	0	0	0	0	71	32	39
	100	100	40.82	59.18	0	100	100	0	0	0	0	100	45.07	54.93
Bunkyo	223	97	37	60	0	4	2	2	0	0	0	106	61	45
	100	100	38.14	61.86	0	100	50.00	50.00	0	0	0	100	57.55	42.45
Taito	146	22	12	10	0	3	0	3	2	1	1	94	46	48
	100	100	54.55	45.45	0	100	0	100	100	50.00	50.00	100	48.94	54.93
Sumida	39	2	1	1	0	0	0	0	0	0	0	25	9	16
	100	100	50.00	50.00	0	0	0	0	0	0	0	100	36.00	64.00
Kohtoh	34	1	0	1	0	2	1	1	0	0	0	28	10	18
	100	100	0	100	0	100	50.00	50.00	0	0	0	100	35.71	64.29
Shinagawa	78	36	18	18	0	2	1	1	1	0	1	10	6	4
	100	100	50.00	50.00	0	100	50.00	50.00	100	0	100	100	60.00	40.00
Meguro	69	35	17	18	0	0	0	0	2	1	1	19	10	9
	100	100	48.57	51.43	0	0	0	0	100	50.00	50.00	100	52.63	47.37
Ohta	90	61	19	42	0	0	0	0	13	3	10	14	4	10
	100	100	31.15	68.85	0	0	100	0	100	23.08	76.92	100	28.57	71.43
Setagaya	90	71	22	49	0	1	0	1	0	0	0	7	4	3
	100	100	30.99	69.01	0	100	0	100	0	0	0	100	57.14	42.86
Shibuya	143	99	29	70	0	0	0	0	0	0	0	31	15	16
	100	100	29.29	70.71	0	0	0	0	0	0	0	100	48.39	51.61
Nakano	40	21	8	13	0	1	0	1	1	1	0	3	2	1
	100	100	38.10	61.90	0	100	0	100	100	100	0	100	66.67	33.33
Suginami	124	87	35	52	0	0	0	0	5	5	0	22	12	10
	100	100	40.23	59.77	0	0	0	0	100	100	0	100	54.55	45.45
Toshima	47	21	8	12	1	8	8	0	0	0	0	17	11	6
	100	100	38.10	57.14	4.76	100	100	0	0	0	0	100	64.71	35.29
Kita	53	32	15	17	0	1	0	1	0	0	0	16	9	7
	100	100	46.88	53.12	0	100	0	100	0	0	0	100	56.25	43.75
Arakawa	20	5	2	3	0	3	1	2	0	0	0	12	4	8
	100	100	40.00	60.00	0	100	33.30	66.70	0	0	0	100	33.30	66.70
Itabashi	14	2	2	0	0	3	1	2	0	0	0	8	3	5
	100	100	100	0	0	100	33.30	66.70	0	0	0	100	37.50	62.50
Nerima	19	14	7	7	0	0	0	0	0	0	0	5	3	2
	100	100	50	50	0	0	0	0	0	0	0	100	60	40
Adachi	17	13	4	9	0	1	1	0	0	0	0	3	1	2
	100	100	30.77	69.23	0	100	100	0	0	0	0	100	33.33	66.67
Katsushika	7	4	2	2	0	1	1	0	0	0	0	2	1	1
	100	100	50.00	50.00	0	100	100	0	0	0	0	100	50.00	50.00
Edogawa	2	1	0	1	0	0	0	0	0	0	0	0	0	0
	100	100	0	100	0	0	0	0	0	0	0	0	0	0
Total	2,191	820	302	517	1	52	29	23	31	15	16	874	359	515
	100	100	36.83	63.05	0.12	100	55.77	44.23	100	48.39	51.61	100	41.08	58.92

Table-4 (Right-half) : Remaining and disappearing situation in each structure

Ward	SRC-structure			others			unknown		
	sub-t	remain	disap.	sub-t	remain	disap.	sub-t	remain	disap.
Chiyoda	49	20	29	6	4	2	57	21	36
	100	40.82	59.18	100	66.67	33.33	100	36.84	63.16

Chuoh	19 100	12 63.16	7 36.84	5 100	2 40.00	3 60.00	77 100	5 6.49	72 93.51
Minato	13 100	4 30.77	9 69.23	4 100	3 75.00	1 25.00	25 100	2 8.00	23 92.00
Shinjuku	2 100	2 100	0 0	2 100	1 50.00	1 50.00	0 0	0 0	0 0
Bunkyo	4 100	3 75.00	1 25.00	3 100	1 33.33	2 66.67	9 100	3 33.33	6 66.67
Taito	4 100	1 25.00	3 75.00	4 100	3 75.00	1 25.00	17 100	1 5.88	16 94.12
Sumida	2 100	1 50.00	1 50.00	0 0	0 0	0 0	10 100	2 20.00	8 80.00
Kohtoh	1 100	1 100	0 0	2 100	2 100	0 0	0 0	0 0	0 0
Shinagawa	1 100	1 100	0 0	1 100	0 0	1 100	27 100	6 22.22	21 77.78
Meguro	7 100	5 71.43	2 28.57	1 100	1 100	0 0	5 100	2 40.00	3 60.00
Ohta	0 0	0 0	0 0	1 100	1 100	0 0	1 100	0 0	1 100
Setagaya	0 0	0 0	0 0	1 100	1 100	0 0	10 100	7 70.00	3 30.00
Shibuya	1 100	0 0	1 100	0 0	0 0	0 0	12 100	1 8.33	11 91.67
Nakano	0 0	0 0	0 0	0 0	0 0	0 0	14 100	0 0	14 100
Suginami	1 100	1 100	0 0	2 100	2 100	0 0	7 100	3 42.86	4 57.14
Toshima	1 100	0 0	1 100	0 0	0 0	0 0	0 0	0 0	0 0
Kita	0 0	0 0	0 0	3 100	2 66.67	1 33.33	1 100	0 0	1 100
Arakawa	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
Itabashi	0 0	0 0	0 0	1 100	0 0	1 100	0 0	0 0	0 0
Nerima	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
Adachi	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
Katsushika	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0	0 0
Edogawa	0 0	0 0	0 0	0 0	0 0	0 0	1 100	0 0	1 100
Total	105 100	51 48.57	54 51.43	36 100	23 63.89	13 36.11	273 100	53 19.41	220 80.59

CHAPTER III

STRUCTURAL CONCEPTS
and
ANALYSIS
of
ANCIENT STRUCTURES

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A CONTEMPORARY CLARIFICATION METHOD FOR DETERMINING EARTHQUAKE RESISTANCE PERFORMANCE IN A TRADITIONAL JAPANESE WOODEN STRUCTURE -EARTHQUAKE RESISTANCE DIAGNOSIS OF *SUOU KOKUBUNJI* TEMPLE-

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ABSTRACT
Earthquake resistance diagnosis was carried out on the Suou Kokubunji temple, which had been taken apart for repairs. This temple is located in Hofu city, Yamaguchi prefecture, Japan, and was rebuilt in 1780. Vibration response analysis was carried out focusing on seismic and wind load by modeling structural elements and quantifying the damping force, which is an important factor. The analysis confirmed that the deformation of the building satisfied the preset criterion of 1/15 the column-rocking angle. Consequently, it was concluded that in the event of a major earthquake, the temple will not collapse and only minor reinforcement will be adequate, thus obviating the need for adding major structural elements to increase its earthquake resistance.

1. INTRODUCTION

The properties of wood members vary widely and no established diagnosis method is available for the diagnosis of earthquake resistance of traditional wooden structures such as temples in Japan. Therefore, in almost all cases, reinforcement with steel frames, which contributes to safety but ignores the earthquake resistance inherent in traditional wooded structure, has been carried out to improve earthquake resistance. On the other hand, considerable progress has been made in numerical analysis such as vibration response analysis and data from element-level research on the resistance of structural members made of wood has been accumulated. This suggests that an earthquake resistance diagnosis method for ancient wood structures will probably be established in the near future.

In this paper, an example of vibration response analysis using the parametric study method is presented to analyze the earthquake resistance of temples of traditional wooden colunn and beam construction. An attempt is made to characterize the earthquake resistance inherent in ancient wooden structures.

2. OVERVIEW OF THE TEMPLE

The Suou Kokubunji temple is a traditional wooden colunn and beam structure that was built in Yamaguchi prefecture, Japan, in 741. The existing main hall of the temple was rebuilt in 1780. The hall is 22.0-m long, 15.8-m deep and 15.8m-height, with a double roof construction using traditional Japanese roof tiles. (Figs. 1, 2 and 3)

The temple does not have structural members such as bracing or walls to withstand an earthquake. Only the rocking resistance of columns and the bending resistance of beams may be considered to provide the effective rocking resistance. No hardware such as nails are used to join wood members. The structure has been formed by skillfully combining bracket complexes that were crafted in a complex manner. The damping force generated by the shake of joints and the friction between wood members is considered to provide significant earthquake resistance to the structure. (Figs. 4 to 6)

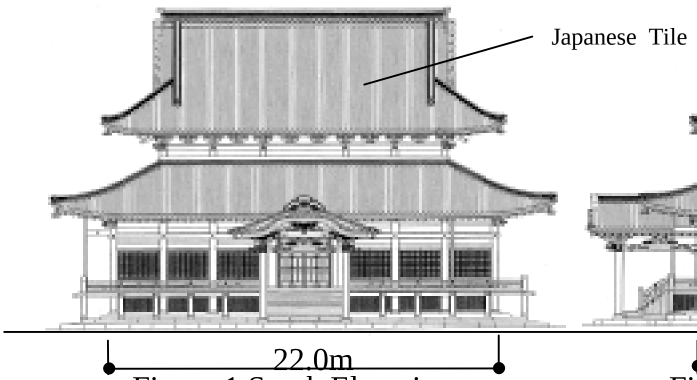


Figure 1 South Elevation

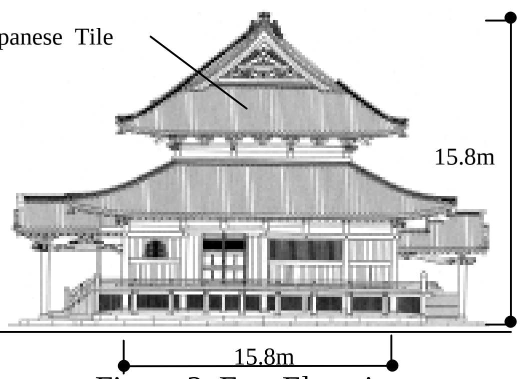


Figure 2 East Elevation

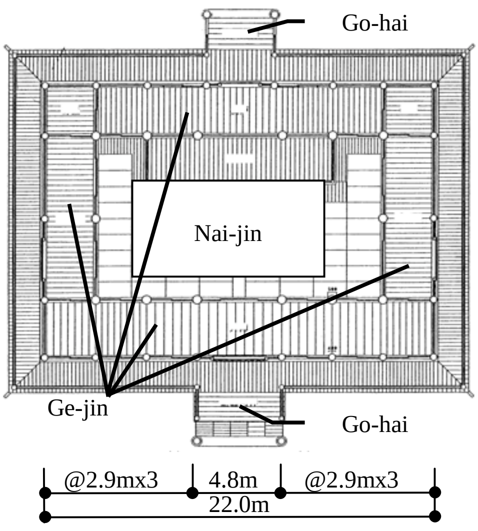


Figure 3 Floor Plan

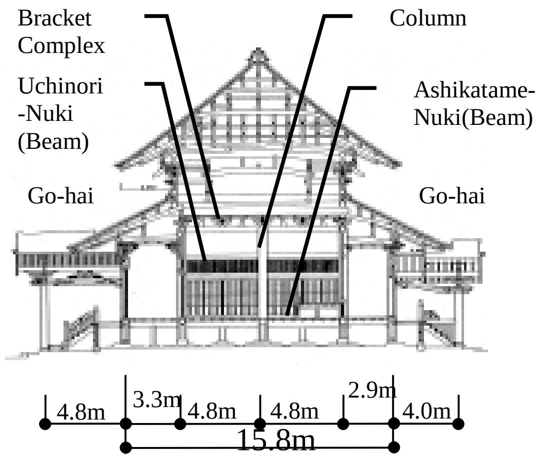


Figure 4 North-South Section

3. INVESTIGATION OF MATERIAL PROPERTIES

3.1 WEIGHT ESTIMATION

The weight of the temple was estimated before conducting the diagnosis. The weight elements were divided into two groups: roofing material including roofing tiles, clay underlay and sheathing boards; and the wood members below the purlin. The density of the wood members was taken as 6 kN/m^3 based on the weight measurement of the wood members that had been taken apart. The weight of the roof becomes about 3.7 kN/m^3 and the weight of the wood members becomes 1.6 kN/m^2 per unit horizontally projected area of the roof. The weight of the roof and wood members was 3600 kN and 1600 kN, respectively, with the roof accounting for 70% of the total weight of the temple structure.

3.2 RIGIDITY AND STRENGTH OF WOOD MEMBERS

Impact tests, embedding tests perpendicular to the direction of the fiber, and bending rupture tests were carried out on the wood members that had been taken apart. Based on the test results, Young's modulus of 8 GPa, compressive yield strength of 7.1 N/mm^2 in the direction perpendicular to that of the fiber, and bending rupture strength of 30 to 60 N/mm^2 were assumed as the values to be used for the analysis.

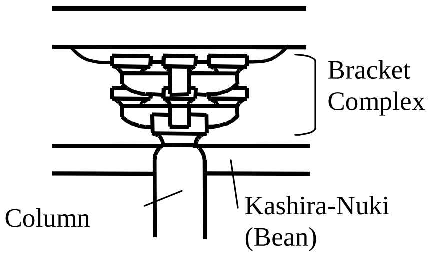


Figure 5 Bracket Complex

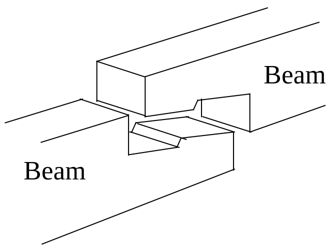
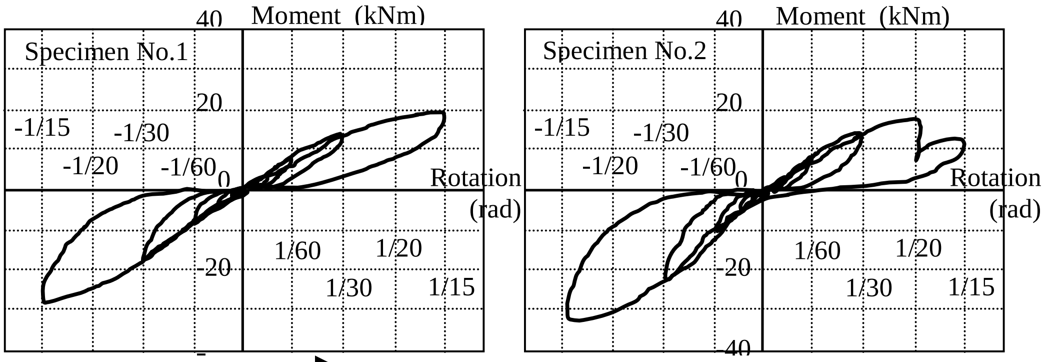


Figure 6 Joint Detail (Ryaku-Kama)

3.3 FULL-SCALE BENDING RUPTURE TESTS OF COLUMN-BEAM CONNECTIONS

Full-scale bending rupture tests of column-beam connections were performed to examine the rigidity and the angle of rotation of the connections. The tests showed a reduction in rigidity due to the cracking in the fiber direction, but no significant reduction in strength for angles of rotation of up to 1/20 for all test specimens. When the angle of rotation reached 1/17, diagonal cracks occurred in one of the six test specimens and its strength reduced to about half. But the specimen retained its strength up to a deformation angle of 1/10. Almost all members showed no strength reduction for angles of rotation of up to 1/15 (Figs. 7). Based on these results, it was assumed that the probability of collapse of the temple due to the failure of column-beam connections was low.



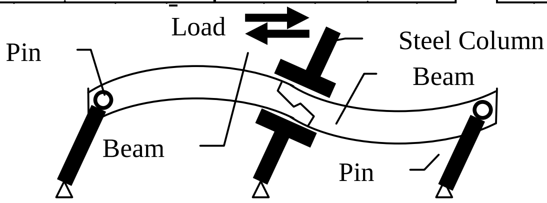


Figure 7
Full-scale Bending Rupture Test
of Column-Beam Connections

4. MODELLING

4.1 ANALYSIS MODEL

In the vibration response analysis model, the mass of the building was represented with two mass points, columns with one column, and the beams at each height with one beam (Fig. 8). The semi-rigidity of the joint due to the embedding of the beam and the deformation of the joint due to the bending of the beam were represented by two rotational springs [4]. The wood members above the column top and the roof were assumed to be rigid because the wood members were densely interconnected.

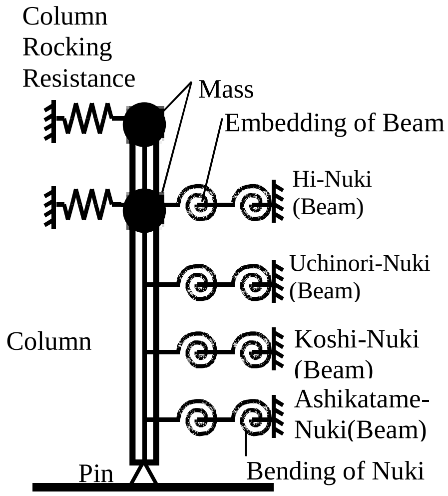


Figure 8 Analysis Model

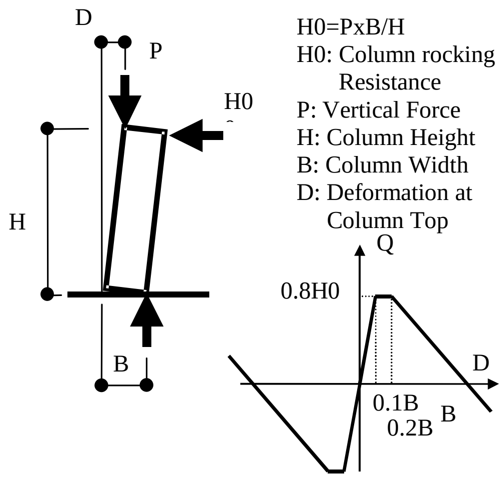


Figure 9 Column Rocking Resistance Model

4.2 COLUMN ROCKING RESISTANCE

Large columns are frequently used in Japanese temple structures. Two types of columns with diameters of 400 mm and 500 mm are used in the Suou Kokubunji temple. Since the column base rests on a stone base, the shearing force generated during an earthquake is transferred only through friction generated by vertical forces. The column top is connected by dowel with bracket complex that is made by combining timbers, and therefore the bending moment is not transferred. Column-rocking resistance occurs due to the shift of the gravity loading point [8] and acts as an effective earthquake resisting element (Fig. 9).

4.3 EMBEDDING

At the column-beam connection, the column transfers compressive forces in the direction parallel to the fiber and the beam transfers them in the direction perpendicular to the fiber. Since wood is an orthotropic material, it is less rigid in the direction perpendicular to the fiber, with a rigidity of about 1/50th of the rigidity in the direction parallel to the fiber. Consequently, embedding occurs in the beam, and the connection becomes semi-rigid. A slip model was used to represent column-rocking resistance, based on the literature [5] (Fig. 10). The column-beam connection of this structure is of a shape called "Ryaku-Kama" (Fig. 6). The full-scale tests and the literature [4] showed that the rigidity and strength of the column-beam connection becomes about half compared to the penetrating (no joint) beam.

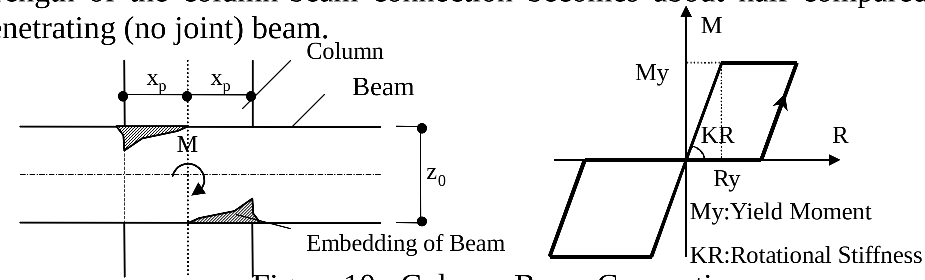


Figure 10 Column-Beam Connection

4.4 DAMPING

The magnitude of damping force has not yet been clarified theoretically. The damping force was quantified by comparing the results of the full-scale vibration tests [10] with the results of the vibration analysis used in this paper. The comparison showed that initial rigidity proportional damping is of the order of 2 to 10% (Fig. 11).

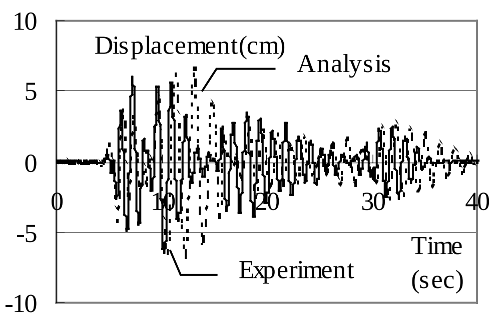


Figure 11
Quantification of Damping Force
(Comparison with Other Tests)

5. SEISMIC FORCE

Seismic force was estimated using the following two methods:

1) Probabilistic estimation of earthquake motion based on earthquake records at the site [3]

   Assuming a return period of 1000 years based on the records of earthquakes that occurred within a radius 200 km around the site in the past 200 years, the maximum ground motion velocity was probabilistically estimated to be about 35 cm/sec. (Figs. 12 and 13)

2) Estimation based on the active fault model [2], [7], [9]

   Assuming the displacement of the faults that have been identified in the vicinity of the site and taking into account the amplifying effect of the surface ground at the site, the maximum ground motion velocity was estimated to be about 40 cm/sec.

   Based on the results of the two estimations above, the maximum ground motion velocity to be used in the vibration response analysis was taken as 40 cm/sec.

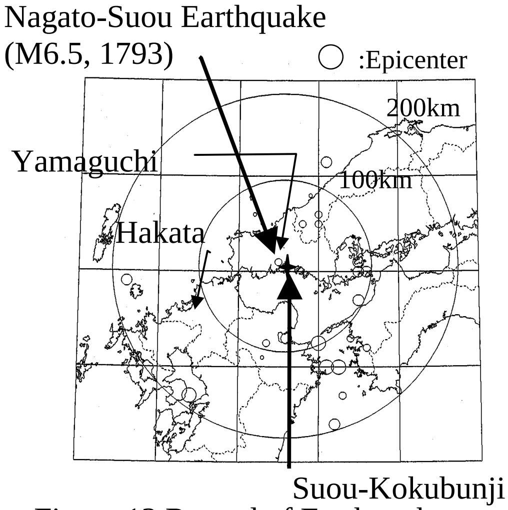


Figure 12 Record of Earthquake

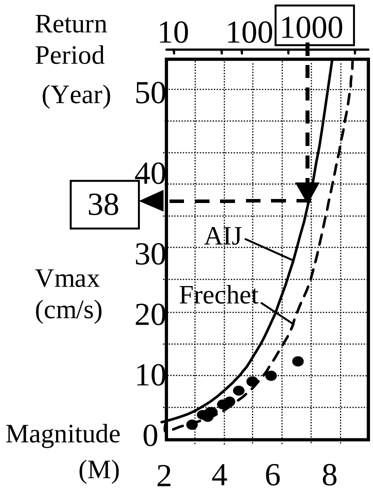


Figure 13 Return Period and Ground Movement

6. GUIDELINES FOR EARTHQUAKE RESISTANCE DIAGNOSIS

The target earthquake resistance of the temple assumed was: the structure should not collapse even if it is partially damaged and loss of life is prevented during an earthquake. This target was expressed in terms of the column-rocking angle, and the criterion for the angle was set as a value less than 1/15, which is the limit at which the column rocking resistance is retained. The safety of the column-beam connection at the column-rocking angle of 1/15 was generally confirmed by the full-scale bending rupture tests. However, some reinforcements are necessary if the column-rocking angle exceeded 1/20, considering that strength reduction was observed for an angle of rotation of 1/17 in one of the test specimens.

7. VIBRATION RESPONSE ANALYSIS

7.1 GUIDELINES FOR VIBRATION RESPONSE ANALYSIS

Five seismic waves were used in the vibration response analysis: three standard waves (El Centro, Taft and Hachinohe) and two simulated waves (Art Wave 1 and 2) generated using the active fault model. They were normalized so that the maximum ground motion velocity became 40 cm/sec.

To take into account variations in the various factors of the wood members, the parameters of the analysis model were varied and an overall judgement of the analysis results was made (Table 1).

Column-rocking resistance showed similar characteristics in directions along and perpendicular to beam. The maximum strength was observed in Case N, the base shear was 0.13 in North-South(NS) direction and 0.11 in East-West(EW) direction (Figs. 14 to 16).

Table 1. Variations in Factors used in the Parametric Study

Factor	Reduction Ratio of Column-Rocking Resistance			Reduction Ratio of Beam Embedding			Ratio of Damping Force		
CASE	RS	N	RL	ES	N	EL	DS	N	DL
Rigidity & Strength of Column-Rocking Resistance	X0.50	X0.75	X1.00	X0.75			X0.75		
Rigidity & Strength of Beam Embedding	X0.40			X0.25	X0.40	X0.50	X0.40		
Damping Force	5%			5%			2%	5%	10%

7.2 RESULTS OF VIBRATION RESPONSE ANALYSIS

The results of the vibration response analysis showed that the column-rocking deformation angle was about 1/25. It was 1/17 when the damping force was small (case DS) of NS direction (Figs. 14 to 16).

The energy input to the structure was mostly consumed in the form of damping energy, which accounted for 90% of the total energy input (Fig. 17).
The natural frequencies of the structure obtained by modal analysis was 1.65 sec in the first mode of vibration (in the beam direction) and 1.56 sec in the second mode (in the direction perpendicular to the beam).

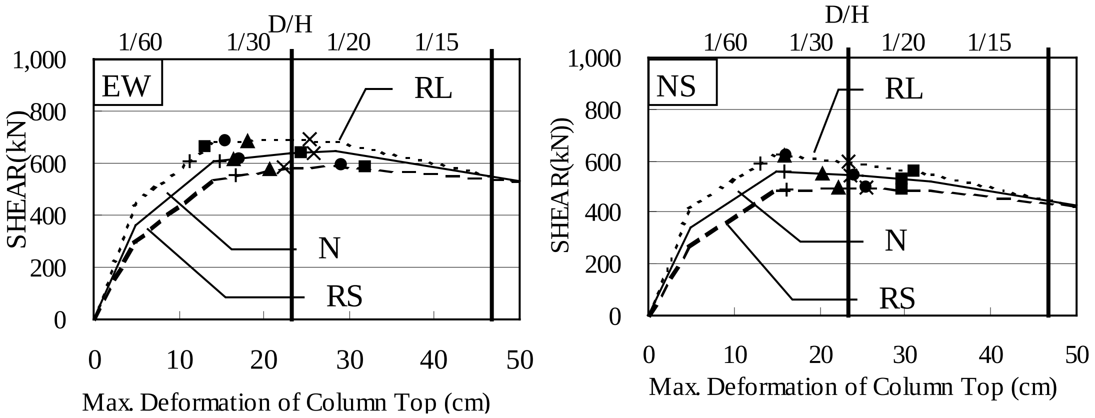


Figure 14 Results of Dynamic Analysis (Factor : Column-Rocking Rsistance)

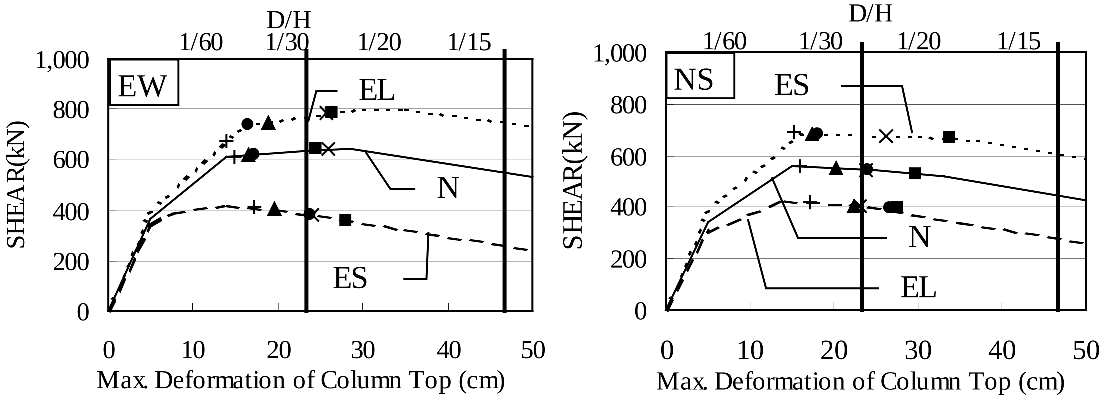


Figure 15 Results of Dynamic Analysis (Factor : Beam Embedding)

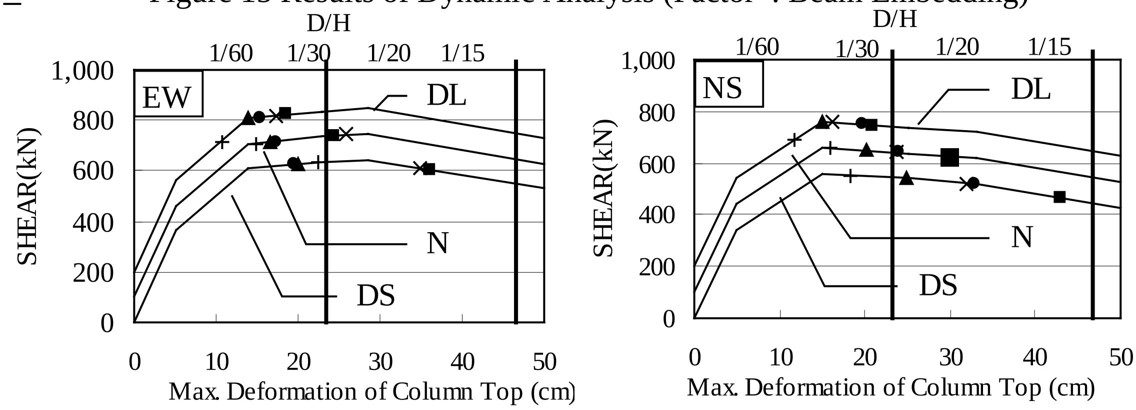


Figure 16 Results of Dynamic Analysis (Factor : Damping Factor)

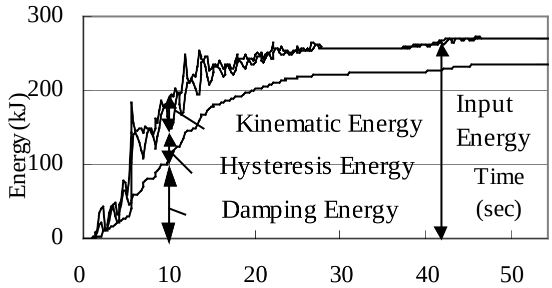


Figure 17
Time History Response
(Ratio of Energy)

8. WIND RESISTANCE

For estimating the wind pressure, the temple structure was approximated as a rectangle so that the same projected area was retained. A time history of wind pressure was generated using the literature [3] and typhoon records. An average wind pressure of 1.3 kN/m^2 (a wind speed of 35 m/sec averaged over 10 minutes) and a maximum wind pressure of 2.0 kN/m^2 were assumed. The vibration response analysis based on this time history of wind pressure showed that the wind pressure exerted on the structure was less than its maximum strength and the column-rocking deformation angle was less than 1/30.

Assuming the occurrence of a typhoon and its course by a Monte Carlo simulation, a 10-minute average wind speed was calculated. The calculation showed that the wind speed was approximately 35 m/sec, thus confirming that the above estimation is reasonable.

9. REINFORCEMENT PLAN

The vibration response analysis predicted that the rocking of the structure is generally less than 1/25. Therefore, it was concluded that no structural elements were necessary to increase the earthquake resistance of the structure. However, the parametric study shows that the deformation angle in one case exceeds 1/20. Therefore, it was concluded that at least minor reinforcements should be provided for Kashira-Nuki (upper level beam) and Uchinori-Nuki (bottom level beam) by connecting the two beams with a stainless steel plate (Figs. 18).

10. CONCLUSIONS

Vibration response analysis was carried out using a parametric study method that takes into account variations in the rigidity and strength of timber and the traditional colunn and beam construction, to analyze the earthquake resistance of a temple with traditional colunn and beam construction.

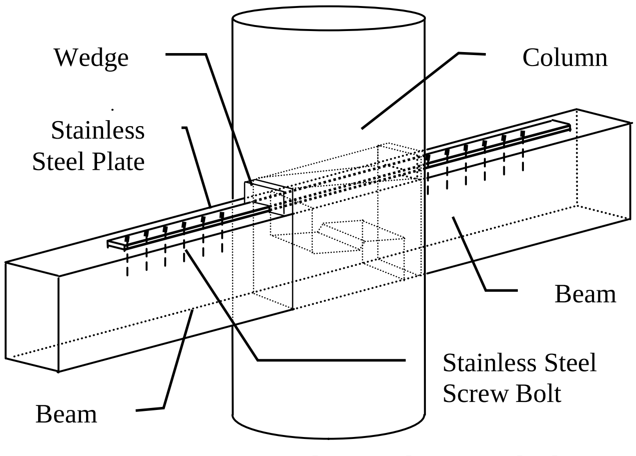


Figure 18 Joint Detail (Stainless Steel Plate)

From the various factors, damping force was quantified based on the results of tests carried out in the past. The analysis confirmed that the column-rocking deformation angle was approximately 1/25 and thus almost satisfied the criterion of 1/20. It also showed that damping force consumes practically all the input seismic energy. Enhancement of accuracy in quantifying the damping force is an issue that needs to be addressed in the future.

The results of diagnosis of the earthquake resistance of the Suou Kokubunji temple showed that the temple can safely withstand a major earthquake without collapse provided rotten wood members are replaced and connections are reinforced to a small extent.

This earthquake resistance diagnosis method can be applied not only to temples but also to all types of traditional colunn and beam structures including pagodas and houses. We believe that this method can make an important contribution to the preservation of historic buildings.

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ANALYSIS OF GOTHIC STRUCTURE

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ABSTRACT

The paper describes structural problems related to different large Gothic cathedrals with the attempt to study them by means of different methods of analysis for masonry constructions. The studies show that the different state of conservation of the examples selected (Tarazona, Barcelona and Mallorca Cathedrals) can be related with the adequacy of their original design.

1. INTRODUCTION

The first attempts to rationally understand the structure of Gothic cathedrals, undertaken during the 19th c., produced a very enthusiastic acceptance of the rationality and sufficiency of their structural arrangement and principles. Viollet-le-Duc writings described Gothic structural ordering as a system strictly based on equilibrium and justified by it. Furthermore, all devices of Gothic structure were laid out so as to contribute to the achievement of an overall ductility. In that context, ductility had to be understood as the capacity of the structure to accept significant deformation without damage or failure. It was this ductility which allowed the Gothic structural components (piers, arches, vaults) to attain their impressive dimensions and their slenderness.

On the contrary, today experts tend to observe Gothic structure from a more cautious point of view, knowing that Gothic structural systems are just an approach, but by no means a full achievement, of true optimal construction. Furthermore, some Gothic constructions (like Sta. María de Vitoria, in Spain, analysed by Croci et al. [1] show remarkable weaknesses related to inadequate aspects of their original design. According to the mentioned researchers, small design or construction imperfections may lead to high deformability and cracking. The study of several important Gothic constructions, including the case of the

Basilica of St. Francis of Assisi after September 1997 Earthquake, has also showed the possible weakness of gothic structures to special actions such as earthquake, intense wind or large foundation settlements. On the other hand, frequent damage patterns have been identified which, in some cases, are considered "chronic", such as the so-called Sabouret cracks typically observed in large groined vaults.

These previous considerations highlight the fact that no general consequences can be drawn with regard to the performance and safety of the structure of Gothic cathedrals. Each case deserves a particular study which considers, in detail, the peculiarities of its structural design and its adequacy for specific materials, dimensions and actions.

Spain's architectural heritage includes a large number of Gothic cathedrals with very different architectural and structural patterns or styles. Some of them - the earlier ones, including some outstanding examples, as Burgos or León Cathedrals- were fashioned within the classical French High Gothic architecture of the 13th c. Other examples, such as those built in the Mediterranean territories of the Kingdom of Aragón during the 14th and 15th c., correspond to a more evolved architecture showing characteristic architectural features and structural innovations.

Three different case studies, analysed by the author, are here considered in order to illustrate the above idea –the fact that each cathedral is a particular case by itself and needs a specific study. The three cases presented –Tarazona, Barcelona and Mallorca Cathedrals- illustrate three different extreme conditions, namely structural insufficiency, structural assurance, and structural audacity. The analyses here referred correspond to the study of the naves of the cathedrals. The study of the cimborios of Barcelona and Tarazona Cathedrals has already been presented [2].

2. TECHNIQUES FOR STRUCTURAL ANALYSIS

The studies are based on the application of two alternate techniques with different degree of sophistication.

Non-linear formulation for 3D framed structures with curved members (GMF). This approach consists of the modelling of the ancient structures of Gothic cathedrals as an equivalent frame with one-dimensional spatial curved elements. These elements are used to describe the piers, abutments, flying arches and vault ribs. For that purpose, a flexibility formulation for 3D framed masonry structures with curved members, based on a generalisation of conventional matrix methods, has been adopted [3].

Consistently with the principles of matrix methods, the flexibility formulation stems exclusively from equilibrium between external and internal forces at any point within an arch or linear member, so that no additional hypotheses over the displacement or stress field are required. Since the movements are fully free (unlike in FEM, where field displacement shapes must be assumed), arbitrarily high concentrated curvatures associated with damage can be reproduced, resulting in a feasible approach for damage localisation, or hinge formation. In order to carry out the non-linear material analysis, masonry is treated as a linear elastic-perfectly brittle material under tension, while elasto-plastic equations are adopted for masonry subject to compression and shear. A Mohr-Coulomb failure envelope is adopted in order to describe failure modes due to combined states of compression and shear.

3D Finite Element Damage Continuum Model. Continuum damage models are particularly useful for the simulation of fragile materials such as concrete, ceramics and stone. In this work two formulations recently developed by Cervera et al. [4] and Oñate et al. [5] for non-linear analysis of concrete, based on the concepts of *damage* above mentioned, have been chosen. These formulations are based on a isotropic damage model with only two internal scalar damage variables to respectively characterise tension and compression damage. This yields a simple constitutive equation which nevertheless enables to simulate all the important aspects of the non-linear behaviour of concrete and masonry, such as the different response under tension and compression, softening due to deformation, and the stiffness degradation due to compression-tension cycles. The damage variables can take values ranging from 0, for undamaged intact material, to 1 (in fact an unreachable bound), for the complete loss of resistance at micro-structural level. The loss of stiffness at each material point is then assumed to be proportional to the damaged parameter, which evolution from zero to one is adequately characterised by an experimental law defined via experimental testing.

3. LAY-OUT OF THE ANALYSIS

The analyses carried out included the assessment of the construction subject to gravity load, settlements and horizontal load due to wind or earthquake. Only the effects of gravity loads and settlements are here discussed.

The study of the structures subject to dead load was carried out by gradually increasing the applied load until reaching its actual value, and then continuing to marginally increase it until causing the failure of the system. In the studies here reported the failure resulted from the development of a ductile ultimate mechanism characterised by a certain distribution of plastic hinges. Similarly, the study of the effect of differential settlements between the piers consisted of increasing the value of the settlement until causing severe damage, and beyond, until simulating the failure of the construction.

In order to carry out the analysis using GMF, a model has been elaborated representing the typical bay of the nave, including the piers, buttresses, flying arches and vaults, all modelled by means of straight or curved linear elements. The vaults of the nave and those of the aisles are modelled as arches with a complex, variable cross section incorporating the transverse rib, part of the membrane of the cross-vault and the masonry backing which exists over the springings of the vault (figure 2). According to the experience of the authors, this treatment can give acceptable results about the equilibrium and strength of the structure in spite of the coarse approach used to model the vaults.

A detailed study of the vaults requires a more sophisticated technique of modelling such as the FEM-based damage model (figure. 1). In this case, thousands of elements are needed for the analysis requiring a much larger computer effort.

The mechanical properties considered for the analyses were decided based on experience available for materials similar to those existing in the buildings. The values initially assumed for the compressive strength are 6,0 MPa and 8,0 MPa for the stone masonries of Tarazona and Barcelona Cathedrals respectively. The effect of possible variations with respect to the assumed values was accounted for by means of a sensitivity-analysis, as is described below.

4. TARAZONA CATHEDRAL

Construction of Tarazona Cathedral began by 1235 over the remains of a Romanesque church. The more ancient parts are the choir and transept, built during the 13^{th} ; the nave and the original cimborio were built during the 14^{th} c. The second, present cimborio was erected later, during the 16^{th} c. The dimensions of the building are rather moderate: the span of the central nave is 7.3 m. and the highness at the keystone of the vaults is about 16.5 m. Due to the material and structural deterioration of the building, a large part of it, including the cimborio and the clerestory walls, was propped on steel frames some decades ago, and the cathedral closed to the public for years. A restoration programme, including repair and strengthening operations, is now being carried out with the aim to eliminate the propping system.

The damage and malfunctioning of the building are caused by some initial design weaknesses, by chemical degradation of the material at the basis of the piers and by overloading caused by the construction of a heavy cimborio during the 16^{th} c. Additionally, during the 16^{th} c. part of the section of the piers of the nave was carved out to make space for a timber choir, causing an increase of the stresses and deformations experienced by them. Today, the limestone original piers and arches of the nave show damage of mechanical origin such as cracking and crushed material in some points. The main lesions observed in the nave are: (1) cracks at about mid-span of the flying arches; (2) in some bays, cracks developed in the transverse ribs of the arches, close to the inner springings; (3) damage in the nervatures due to excessive compression at the inner springings of the aisles; (4) vertical cracks separating the masonry backing of the vaults from the walls, and (5) and a large crack at the key of the main transverse arches.

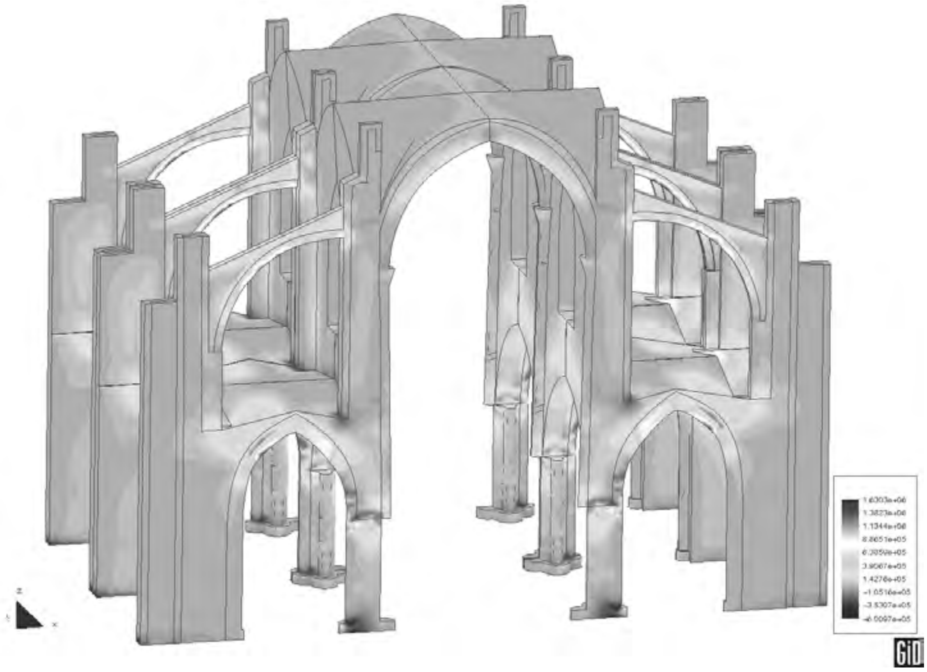

Figure 1 – FEM model elaborated for the study of Tarazona Cathedral by means of continuum damage model. The distribution of damage in dead load is represented in grey scale.

The results obtained for dead loading using the two methods (FEM damage model and GMF) are consistent, in terms of distribution of stresses and cracking, with the lesions observed in the real construction, although some differences exist, as can be inferred from figures 1-2. In overall, the GMF model seems to predict in a more satisfactory way the damage observed in the real construction, in spite of its simplicity compared to the continuum damage model. However, the FEM model succeeds in locating the main crack of the aisles not at the crown but closer to the inner springing, as observed in the real construction. This lesion is due to the fact that part of the thrust coming from the pier is received by the backing of the vault, causing flexion to it. This is a 2D effect which can be hardly reproduced by means of the linear elements used in GMF. In any case, both methods coincide in demonstrating that the existing lesions can be explained because of the dead load itself. Possible settlements, as showed by the analysis, may have contributed significantly to worsen the state of the construction by extending the mentioned damage. Both techniques of analysis agree as well in predicting a precarious condition of equilibrium. According to GMF, failure would occur at 110% of the dead load, while the FEM damage model predicts the failure for 130% of the dead load. Given the existing damage and given the very small marginal capacity predicted, the need to prop the construction until the implementation of some strengthening measures seems clear. The parametrical studies showed that a possible increase of the compressive strength of the materials would not provide a significant increase of the ultimate capacity. However, a moderate decrease of the compressive strength leads, according to the study, to a significant reduction of the total dead load which can be resisted by the system.

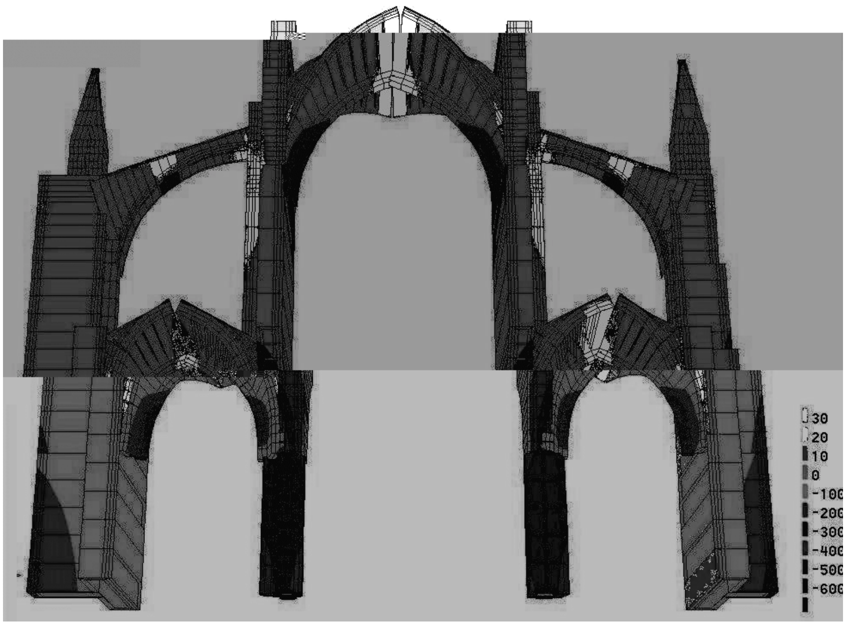

Figure 2 – GFM model prepared for the study of Tarazona Cathedral. The zones experiencing cracking in dead loading are represented in white.

5. BARCELONA CATHEDRAL

Construction of the naves of Barcelona Cathedral was begun in 1298 and lasted for more than a century. As usual, the choir was constructed first, being finished in 1327, while the construction of the entire nave continued until 1417. In 1422 work stopped, leaving the cimborio unfinished and a provisional wall closure as a façade. The building has a three-nave plan (the nave and two aisles) although, as a consequence of its particular design, it appears to enclose two additional aisles. This particular effect is caused by the inclusion of the imposing buttresses in the interior space between the side chapels (figure 3). The nave spans 12.80 m and has a maximum high of 25.6 m. The span of the side aisles is equal to one half the span of the nave. The rise of the vaults at the side aisles, of 20.5 m, begins close to the springings of the central vaults. Thanks to this particular arrangement, the lateral thrust of the central vaults is efficiently carried to the buttress by the lateral vaults so that actual flying arches are in fact not needed. The overall system shows –as demonstrated by the analyses- large robustness thanks to the imposing buttresses (with a base of 7.4 m of length equal to 58% the maximum span) and the optimal structural arrangement. The existing flying arches are but draining devices with no structural role.

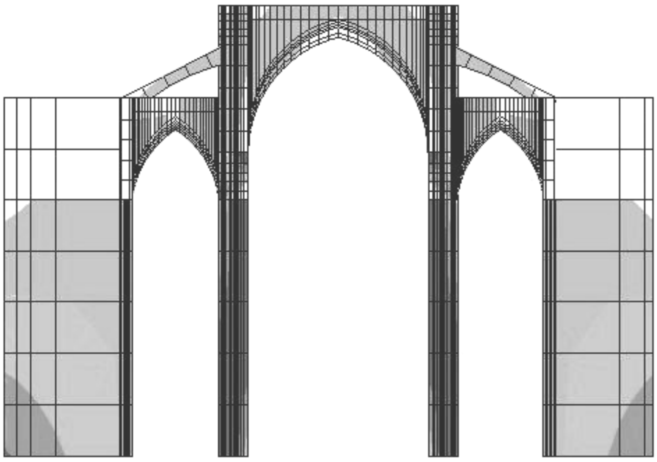

Figure 3 – GMF model elaborated for the structure of Barcelona Cathedral

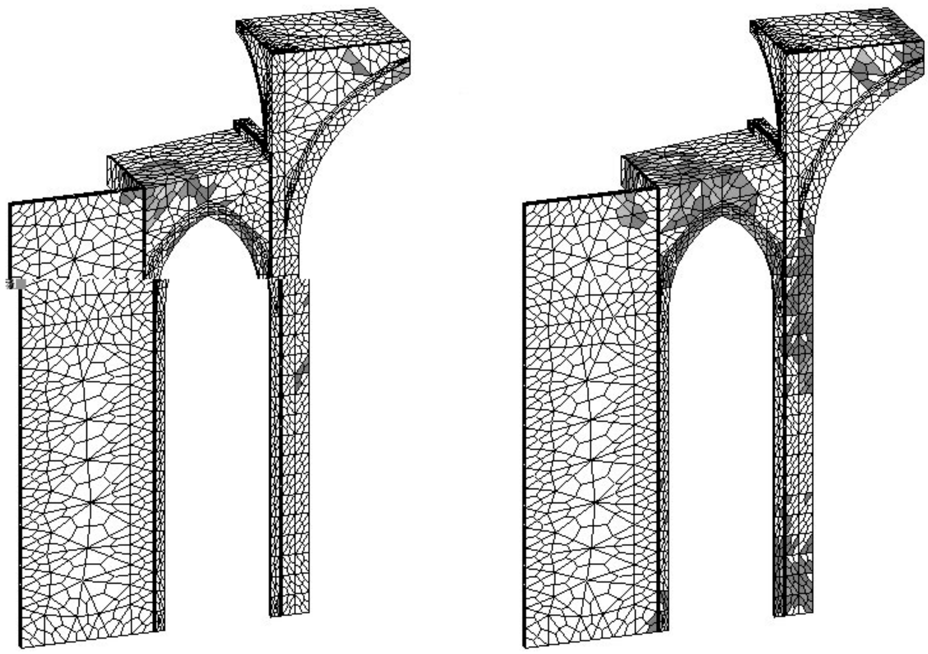

Figure 4 – FEM model of the structure of Barcelona Cathedral. The distribution of damage is represented, in grey scale, for dead load (left) and for a condition close to the failure (right) caused by gradually increasing the dead load.

The analyses predicted that the structure does not experience any significant damage when subject to dead load. Actually, no significant damage has been detected in the structure. On the other hand, further increases of load are allowed until causing the failure at 200% of the dead load (200% and 210% according to GMF and FEM-continuum damage model respectively). Although these value does not have a clear meaning (not allowing an identification as a truly “safety factor”), it gives idea of the resistant sufficiency of the construction.

Figures 2-3 show the distribution of the tension damage (cracking in GMF) failure as predicted by the two methods. Damaged zones in tension first appear at the crown of the transverse arches of the nave. As additional load is progressively applied, damage tends to cover larger regions of the structure. Further damage focuses are observed at the crown of the aisle vaults, at the haunches of the aisle arches and at the bases of the piers. Compression damage keeps almost null but for very high levels of load applied. As can be seen in figure 5, damage tends to concentrate in the regions where severe cracking also appears in the GMF model associated to the development of plastic hinges. The parametrical studies showed that a moderate decrease of the assumed compressive strength of the materials would not provide a significant variation of the ultimate capacity.

According to the analyses, the construction can resist a differential settlement of larger than 3 cm without experiencing severe damage. This illustrates the extreme ductility of the system.

6. MALLORCA CATHEDRAL

The nave of Mallorca Cathedral, initiated in 1350, can be placed among the more outstanding Gothic Cathedrals ever built thanks to its grandiose dimensions and the extreme slenderness of the structural elements. Its 44 m vault keystone height is only exceeded by the choirs of Beauvais and Cologne Cathedrals, while the free span of 17,8 m of its main arcade is only surpassed by the 21,8 m wide unique arcade of Girona Cathedral. The main piers supporting the vaults and clerestory walls have octogonal section with diameter of 1.6 or 1.7 m. The slenderness of the piers, reaching a ratio of 13.8 between diameter and high, constitute the more unique and audacious aspect of the building and contributes largely to a greater sense of internal spaciousness; in the case of other medieval cathedrals, this value stays between 8 and 9 (9.7 for the piers of the choir of Beauvais Cathedral). These extreme dimensions and slenderness are by all means involved in today’s large deformations which are experienced by the piers and, overall, by the construction. A certain set of vertical cracks is also observed at the base of some of the piers. Because of those alterations, the structure desers careful assessment. The structural analysis, based on the mentioned techniques, is now being carried out and results are expected in the immediate future.

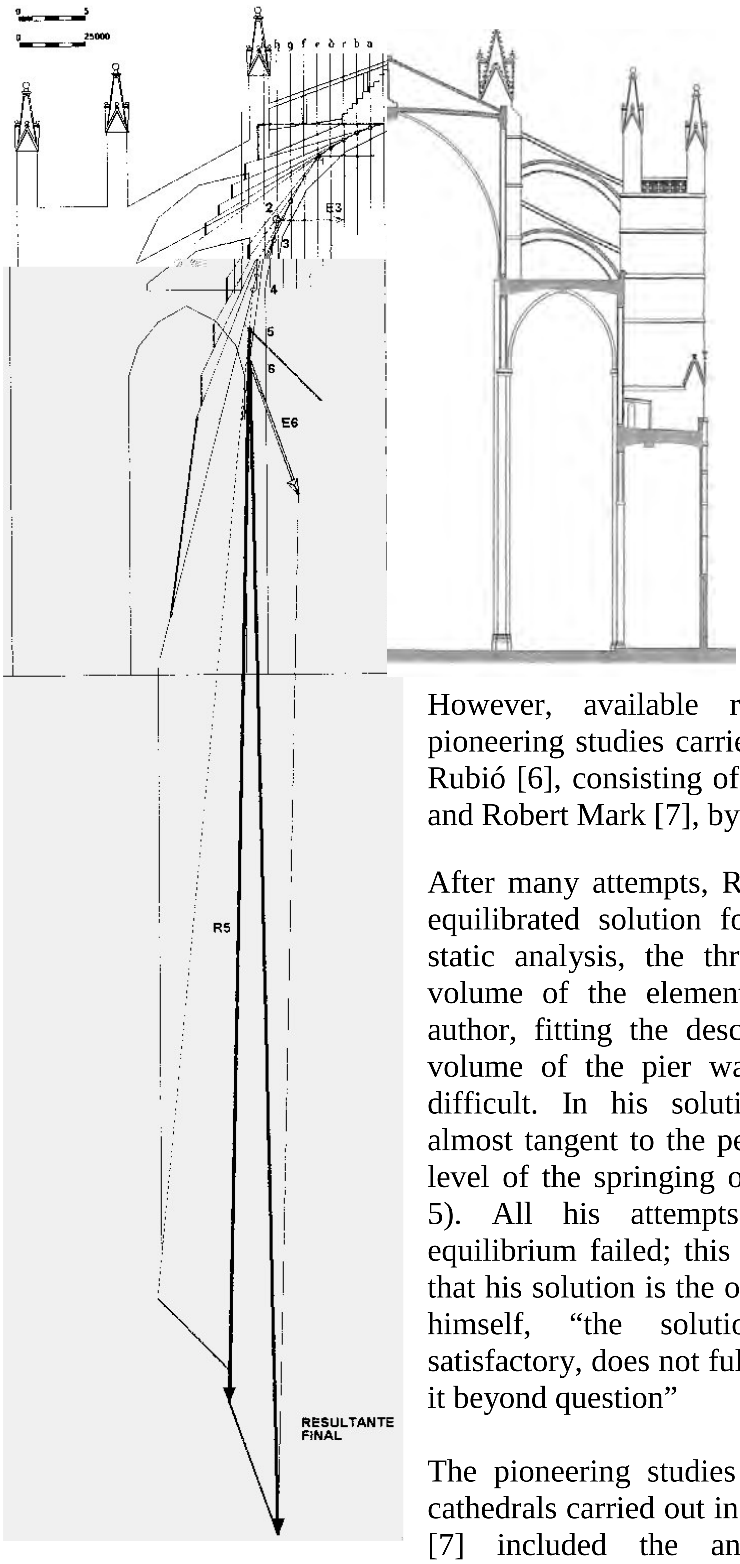

Figure 5 – Section of the nave of Mallorca Cathedral (right) and reconstruction of the analysis by Rubió (left).

However, available results exist thanks to pioneering studies carried out by architect Josep Rubió [6], consisting of a detailed static analysis, and Robert Mark [7], by photo-elasticity.

After many attempts, Rubió was able to find an equilibrated solution for which, as required in static analysis, the thrust remained within the volume of the elements. As explained by the author, fitting the descending thrust within the volume of the pier was revealed as extremely difficult. In his solution, the thrust becomes almost tangent to the perimeter of the pier at the level of the springing of the lateral vault (figure 5). All his attempts to find an alternate equilibrium failed; this fact, still, does not mean that his solution is the only possible. As stated by himself, “the solution obtained, even if satisfactory, does not fully content the spirit nor is it beyond question”

The pioneering studies on the structure of Gothic cathedrals carried out in the 70’s by Robert Mark [7] included the analysis by photo-elastic modelling of many emblematic Gothic constructions. Given its structural interest, the case of Mallorca Cathedral was also considered and analysed using the same technique. The analysis afforded interesting conclusions about its structural features and response subject to gravity loading and wind. Interestingly, some of the conclusions reached by Mark are not in agreement with those drawn by Rubió. According to Mark [7], the photoelastic study predicts a very uniform state of compression in the piers under dead-weight, indicating that the amount of bending is so negligible as to be unique among the many Gothic churches discussed by the author.

It is expected that the analyses now in course provide additional insight on the equilibrium of the building. In particular, the condition of the building should be placed between the two opposites -the daring, extreme equilibrium envisaged by Rubió, or Mark's more uniform, convenient state of forces.

7. FINAL REMARK

The three different case studies here presented illustrate the very diverse conditions which may be experienced by a Gothic construction as a consequence of their original structural design. These conditions range from severe structural disorder (Tarazona Cathedral) to almost full material and structural integrity (Barcelona Cathedral). Eventually, the case of an audacious structure (Mallorca Cathedral) may challenge the capacity of the analysts to draw conclusions about the adequacy of the design and the actual significance of its deformations and lesions.

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ON LIMIT ANALYSIS OF GOTHIC VAULTS

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ABSTRACT

As a result of a research project dealing with the analysis and repairs of gothic rib vaults, several limit states have been established for these structures. These limit states depend basically on geometrical dimensions of vaults and supporting arches.

1. INTRODUCTION

Supported by the research program "Promoción General del Conocimiento", a study on limit analysis of cross vaults is being performed at the Universidad Politécnica of Madrid. This limit analysis has been developed following two methods: the classical limit analysis of masonry arches, and the procedure proposed by Livesley [1], in which linear programming is applied to the equilibrium of a structure formed by rigid blocks. The result of these two methods agree very closely. Before describing the results obtained, general principles used to apply limit analysis to masonry structures must be exposed.

2. LIMIT ANALYSIS APPLIED TO MASONRY VAULTS

Limit analysis has been applied to masonry vaults with the following peculiarities:

1) Masonry can resist a low tension stress (of about 0,30 Mpa) depending on the orientation of the joints [2]. Nevertheless, any crack at the structure -for instance, one produced by temperature or by a settlement- can propagate easily transforming the structure into a very different one. An equilibrium state involving tension stresses can therefore be described as a "fragile" or

non stable state. Anyway, it must be noted that many stone structures hold during centuries working clearly in tension.

2) Compression stresses in masonry structures are usually very low, but that is not the case if the structure is submitted to bending.
As it is well known, bending strength is only possible (Fig, 1) if an axial force acts at the same time. As shown in a previous paper [3], the

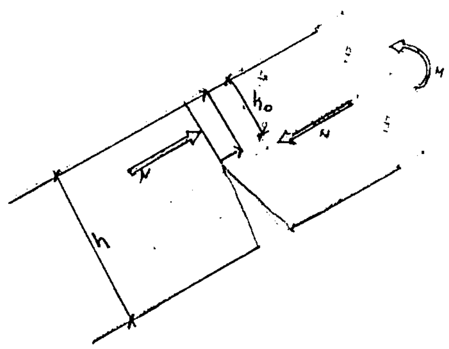


Fig. 1

condition of bending resistance without tension stresses is:

$$h \geq h_0 + \frac{2M}{N}$$

where h is the depth of the structure, h_0 is the depth strictly necessary to resist the axial force without bending, M is the bending moment, and N the axial force. A masonry structure can therefore fail by three causes: because $h<h_0$, because $h< \frac{2M}{N}$, or by a combination of the former two causes. The second one is the basic condition for the stability of a masonry structure, when it works in bending.

3) As has been previously shown [3], the above condition must be established by obtaining the relationships between bending moments and axial forces along the structure, and not by obtaining the thrust line. This has been done by Livesley's method and, alternatively, by a limit state method proposed by Felix Candela for concrete arches.

Candela's method can be described as follows: Suppose an arch submitted to loads producing a global bending moment M_v. If the arch is symmetric, and yield bending moment M_p is the same one at the crown and at the abutments, equilibrium at the crown of the arch can be written (Fig. 2) as:

$Mp = M_v - H.f + Mp$ and we obtain:

$H = \frac{M_v}{f} = H_{3a}$ that is, the thrust is that of a three hinged arch with the same load and the same form. Bending moment (M) at a point of the arch is therefore:

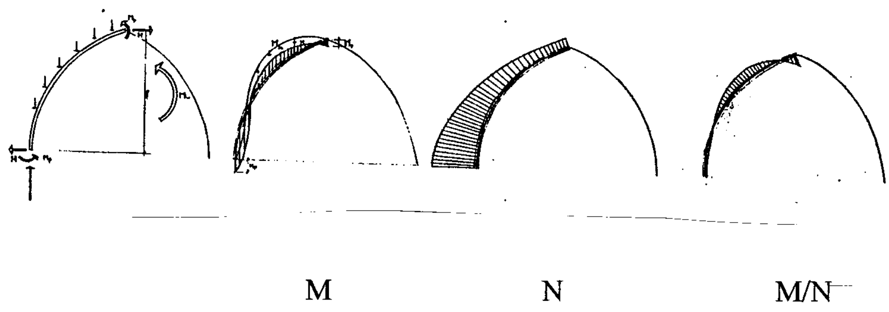


Fig. 2

$$M = M_v - H_{3a}\, y + M_p = M_{3a} + M_p$$

That is, the bending moment of a three hinged arch M_{3a} plus a constant value M_p. Bending moment diagram can be obtained easily sliding vertically the bending moment diagram of a three hinged arch. As axial force diagram can be obtained readily by: $N = T_v \sin\gamma + H_{3a} \cos\gamma$, where T_v is the global shear force of the structure and γ the slope of the arch, the yield diagram of bending moment can be obtained by equating the value of $\frac{M}{N}$ at five points of the arch.

If the moment at the crown M_p^1 is supposed different of the moment at the abutments M_p^2, the value of the thrust can be corrected making:

$$H = H_{3a} + \frac{M_p^1 - M_p^2}{f}$$

The results of Candela's method agreed very closely with that of Livesley's one, in all the cases studied (Fig. 3).

Candela's Method

Livesley's Method

Fig. 3

4) Previous to the definitive cracked state of a vault a membrane state exists in most of these structures. This membrane state is important for two reasons. First, because it can produce a failure at the structure supporting the vault, and therefore can be a first cause of the collapse of the vault when it is in a "fragile" state. Second, because it is the primary cause of cracking, and, therefore, of the transformation of the vault into a different structure.

5) If all the vaults studied have the same thickness, self weight load does not depend on the volume of the structure. This means that relation between bending moment and axial force depend only on the span of the structure. Therefore rules of geometrical proportion can be applied to these vaults.

These principles applied to gothic rib vaults have given the following results:

3. MEMBRANE STATE OF GOTHIC VAULTS AND LIMIT STATE OF FORMERET ARCHES

Membrane state of a gothic vault has been obtained in a previous paper [4]. In this study, vaults are supposed to be supported at ribs and at formeret arches, and ribs are supposed to be free of bending. As the results of this membrane state agree with that obtained by means of Finite Element Method, it can be supposed that state to be the primary one in gothic vaults. Membrane stresses are of tension (Fig. 4) at upper part of the vault.

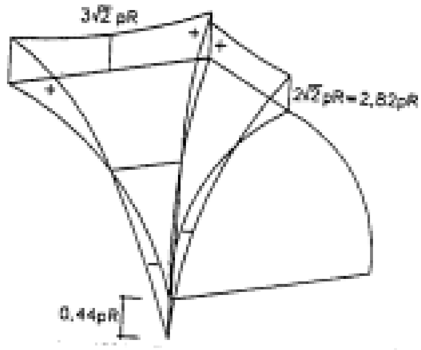


Fig. 4

If the spans of the vault are not greater than 7m. x 7m. in a rib vault square in plan, or 12 x 6,5 m. in a rectangular rib vault, tension stresses are not greater than 0,30 Mp_a. That means that a vault of those dimensions can hold in a "fragile" state. In fact this is the case of many small gothic vaults.

Membrane state produces significant loads at formeret arches (Fig. 5). If Candela's method is applied in order to analyse its bending resistance, it can be found that it needs to be built with a ratio between the depth h and the span L of about $\frac{L}{h} = 7$. This is the usual proportion in most of formeret arches, with the exception of the former ones of Beauvais Cathedral. Membrane state can therefore produce a limit state on formeret arches, that could be one of the causes of the first collapse of Beauvais.

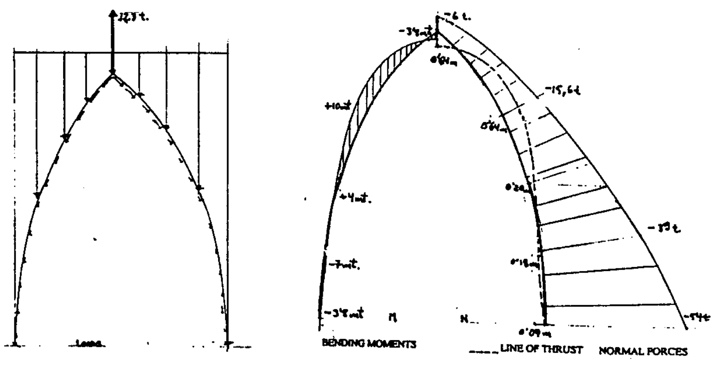


Fig. 5

.a)Loads acting in the plane of formeret arches.b)Efforts at formeret arches.

4. VAULTS WITH TRANSVERSAL CRACKS. LIMIT STATE OF THE VAULTS

If the vault is square in plan, membrane state does not produce a global thrust at the abutments [4]. If the vault is rectangular in plan only a small thrust of about 0,4 PR^2 -where P is the load at the vault and R de radius of formeret arch- appear. However a continuous thrust acting in the perpendicular direction of the plane of formeret arches produce a global bending moment of about 1,85 PR^3. (Fig. 6)

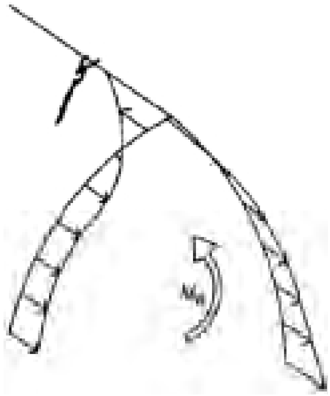
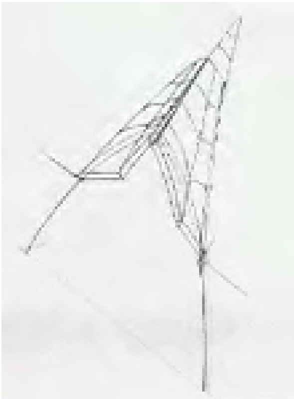


Fig. 6

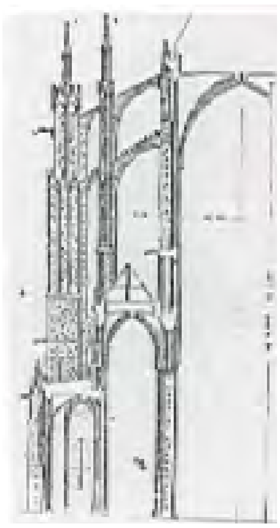
This bending moment tries to separate formeret arches from the vault, and must be the cause of the transversal cracks that appear at the crown of the vault known as “Sabouret cracks”. These cracks transform the structure into two different ones: one composed by part of the vault joined to formeret arches and another composed by the rest of the transversal vault with one free edge. Formeret arches are in this state submitted only to its self weight. Therefore this state of stresses is safest for these arches. For the rest of the vault membrane state is no longer possible [4], and therefore bending moments must appear at the vault in addition to axial forces.

It can be imagined many ways of transmitting loads to the abutments. The simplest one is to suppose the vault working as a number of vertical arches supported at the ribs. Candela’s and Livesley’s methods have been applied to this particular case. The results prove that these arches collapse under its self weight if the vault has a span of 10 m., and only are stable all the arches for spans of approximately 5,4 m. (Fig. 7).

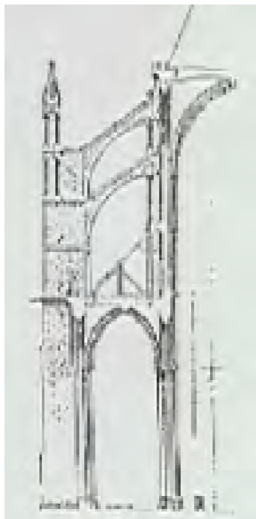
Fig. 7



It suffices therefore the existence of two contiguous transversal cracks to produce the partial collapse of a vault, if its span is larger than the above limit. All this explains the collapse of all the great vaults square in plan constructed in the early gothic (St. Germer en Fly and St. Denis, for instance), whose dimensions made possible the existence of cracks in membrane state. In fact, during the XII[th] century only sexpartite vaults were constructed and in the XIII[th] century, nearly all the great vaults were rectangular in plan with transversal vaults spanning 8 meters as a maximum. The addition to nearly all this vaults of upper flying buttresses (Fig. 8) can also be explained in this way: a passive thrust acting at the top of the vault produces a compression that prevents that more than two contiguous cracks appear. In fact most of vaults with cracking problems -as those of Sta. Maria of Vitoria- have only the lower flying buttress.



Beauvais



Amiens

Fig. 9

5. LIMIT STATE OF RIBS AND PIERS

If only a complete Sabouret crack is produced -as happens in vaults of Amiens Cathedral (Fig. 10)- the structure can be stable provided that the measures of the vault are within the safe limit. In this case, limit state of the structure can be that of the ribs. Anyway ribs cannot be studied as supporting vertical

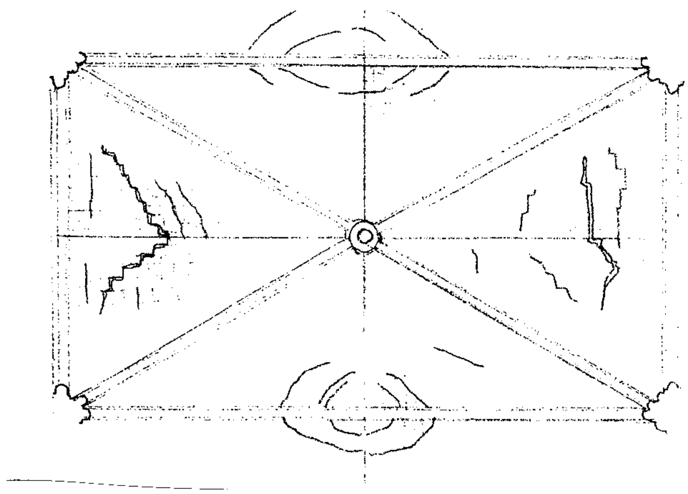


Fig. 10. Cracks at Amiens Cathedral

arches because the joint between vertical arches and ribs implies a Coulomb friction. As Livesley has shown [1], this mechanism does not obey normality rule and therefore limit analysis theorems cannot be applied. Horizontal forces must be added to that joints in order to reduce Coulomb friction.

Finally, it can be imagined vaults with safe proportions and perfectly stabilized by ties and flying buttresses with other limit states. That is the case of the deformation of the piers, possibly produced by the global bending moment of the transversal vaults. In many cases -as those of St. Quentin, St. Jean du Marche at Troyes, or Beauvais Cathedral- deformations of the piers are so great, that a collapse can be produced by non linear equilibrium. This is possibly the cause of the collapse of some vaults of Sevilla Cathedral in the XIXth century.

6. CONCLUSIONS

Depending on the geometrical proportions of vaults, formeret arches, ribs and piers, different limit states can appear at gothic vaults. The aim of a study on limit states for these structures must be to define the geometrical features of each limit state, in order to make the necessary repairs for each particular case.

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STUDY ON OLD MASONRY STRUCTURES IN BRICK VAULTS

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ABSTRACT

At the time of 17th to 19th centuries, comprising the Baroque era and, later on, the epoch of the Eclectic architecture, across the whole Europe a lot of civil buildings were erected in masonry structures with brick vaults. Brick vaults of several forms and structures can be often found by multi-storied buildings, especially at the basement and ground floor level. The intermediate slabs are of timber or mixed (metal and brick) structure, while the top floor, under the roof, usually consists of timber slabs. These buildings, many of them objects of architectural heritage, have suffered different transformations and damages during the time. The most representative brick vault structures and their structural deficiencies and damages are put in evidence and exemplified. Solutions for strengthening are presented.

1. INTRODUCTION

Studies on historical buildings should have two main sources. First of all, actual expert investigations on buildings and, secondly, the technical literature, especially old documents, drawings and books. Therefore, based on a longtime engineering experience in this field, as well as theoretical investigations, the authors try to define, classify and characterize the different types of historical brick vaults representing the main supporting structure of historical masonry buildings.

In a former study [1] the authors have presented a general method for determining the technical state of the constructions being exemplified by the case of a typical house of middle-class citizens of the Baroque era. We have proposed a list of faults and the corresponding penalizing points for one- and two-storied masonry buildings. This has to be completed and concluded in the future.

Based on these former studies, as well as on some further investigations, rehabilitation projects [2], [3], [4], [5] and other references [6], [8] we propose to point out the most characteristic structural deficiencies and damages of the mulistorid historical buildings, as well as their causes. We are talking about those deficiencies and damages which may put in danger the structural resistance and stability of the building, or could lead to such a state, in other words, which could generate collapse. Discussion on the strengthening of masonry structures in brick vaults is presented.

2. CLASSIFICATION AND DESCRIPTION

The vaulted brick structures for civil buildings can be classified in three main types:

(1) Brick vaults bridging relative large spans, supported by brick walls or/and boundary arches - we will call them "brick vaults";

(2) Slab structures consisting of small barrel vaults of brick, supported by metal beams - "small brick vaults";

(3) Other mixed structures of brick vaults or arches and metal profiles of cast iron or mild steel, sustaining or helping each other in overtaking the loading or discharging a certain constructive element - "mixed structures".

2.1. Brick vaults

The brick vaults structures bridging relative large spans, usually equal to the dimensions of the rooms, characterize the architecture of the Baroque era at the time of 17th and 18th century, but they also appear later, during the 19th century covering the basement and sometimes also the ground level spaces.

The geometrical form of the vaulted element mainly depends on the plan area to be covered and the spans to be bridged. In the same time the weaving way of the bricks may be of several types. The most often used types of brick vaults are:

A) Cylindrical or barrel vaults, which can be built using several ways of weaving of the bricks; For instance: the usual cooper's mode of binding (Fig.1a), barrel vaults built in circular layers (Fig.1b) or in swallow-tail bound vaults (Fig.1c).

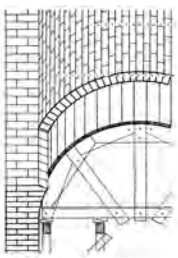


Figure 1b

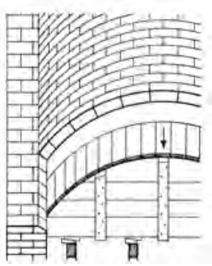


Figure 1b

Figure 1c

B) Cross- or groined vaults, also with several possibilities of the masonry binding, like the former: the usual cooper's mode (Fig.2a), swallow-tailed (Fig.2b) and in circular layers built (Fig.2c) cross-vaults.

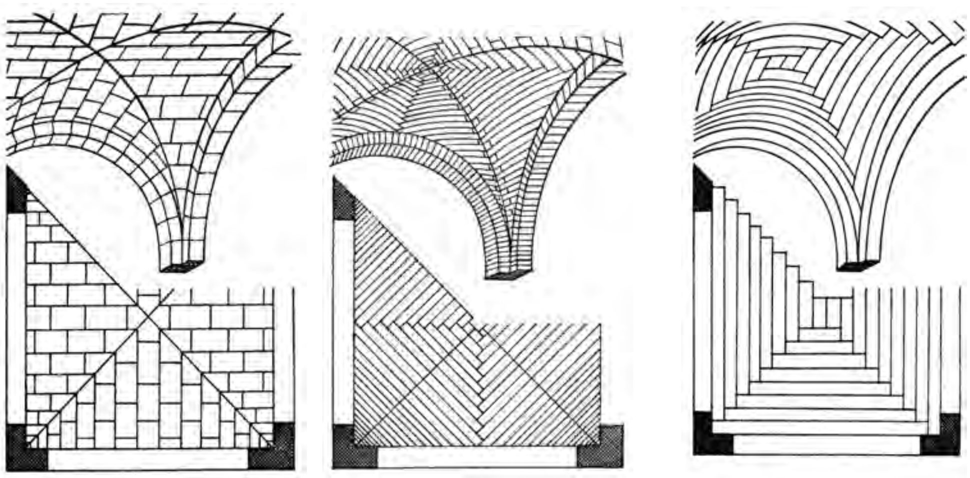


Figure 2a Figure 2b Figure 2c

C) Cloister vault (Fig.3) with usual technique of weaving;

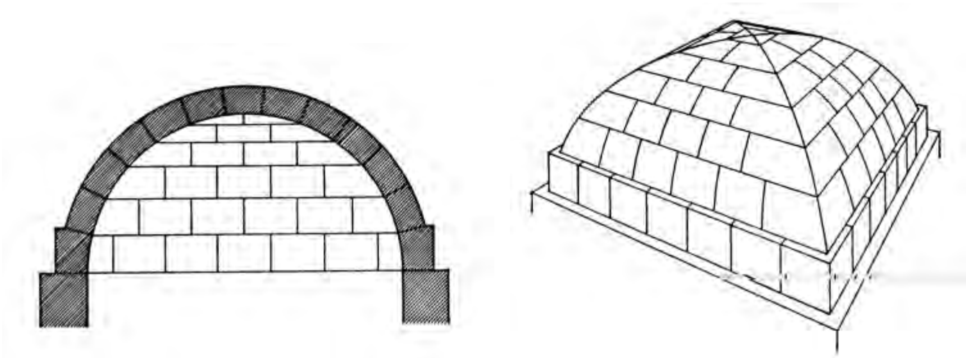


Figure 3

D) Elliptical vault built on rectangular plan (called also "Bohemian vault") presented in the Figures 4 and 5, or on elliptic (circular) plan; their geometrical form is of elliptic paraboloid. They are supported along the boundaries by walls or/and arches, respectively by basic rings and walls or/and inclined arches. It is to be mentioned that during the 19th century the interior vertical supports (pillars) of the Bohemian vaults were often built in cast iron (like in the Figure 5).

E) Other forms resulted by intersections and combinations of different surfaces.

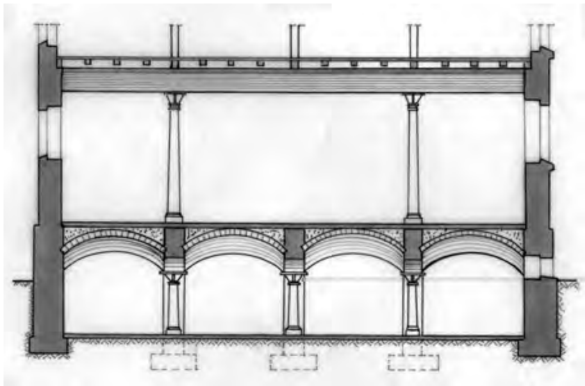
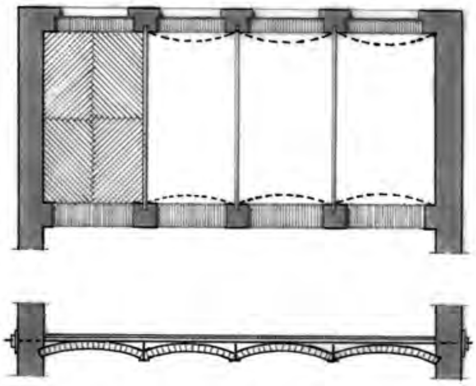


Figure 4 Figure 5

2.2. Slabs of small brick vaults

This type of slab was preferred for all kind of multi-storied residential and public buildings during the 19th century. They consists [6] of small barrel vaults (Fig.6) supported by beams of cast iron or laminated steel, often rail profiles (Fig.7).



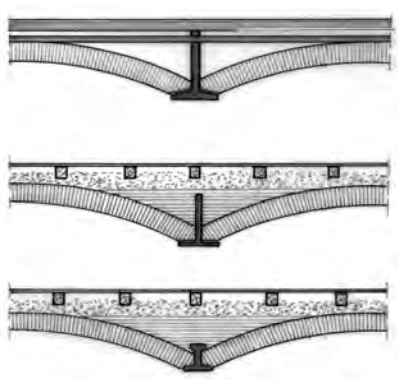
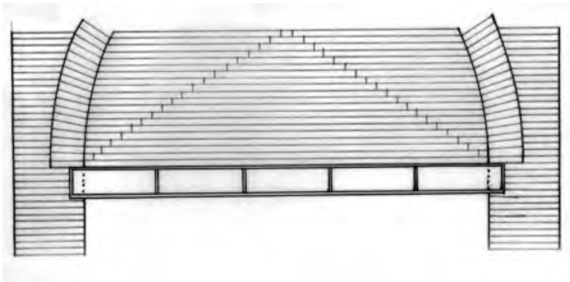


Figure 6 Figure 7

The beams are supported by the main vertical walls provided with special supporting pieces of metal or stone which distribute the vertical forces. In some cases, when the marginal walls were not able to take over the horizontal thrusts, tie bars are provided. The small brick vaults are bridging spans of 1,00 to 2,00 meters and usually have an arch thickness of half brick.

2.3. Beams discharging arch systems

By many structures built during the 18th and 19th century, built in the wall discharging brick arches were used [6]. In order to guard the relatively large-spanned beam from the whole weight of the upper wall an arch of the same span could be built over (Fig.8). Such discharging arch systems can be easily observed on the facade of some multi-storied buildings (Fig.9).



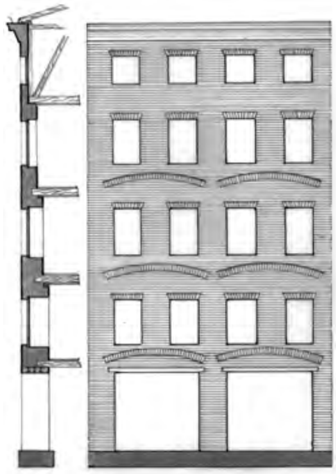
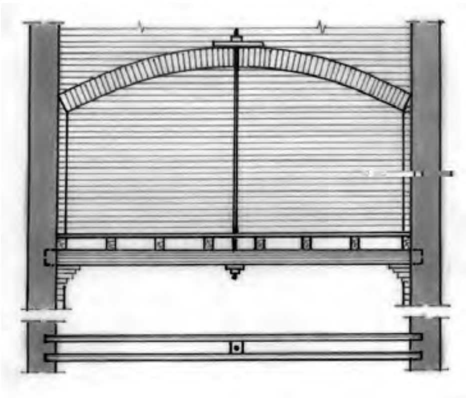


Figure 8 Figure 9

Discharging arches were also used in order to fire slab beams of the weight of the upper interior wall (Fig.10a) or aid brick lintels in overtaking vertical dead loads (Fig.10b).



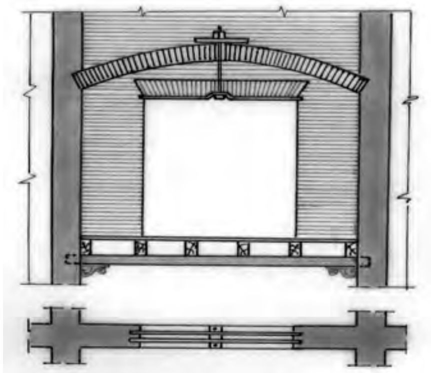


Figure 10 a Figure 10 b

It is important to study old masonry structures for at least two reasons: it offers concepts and details about the building to rehabilitate and rehabilitate and also give suggestions for the possible interventions.

3. DEFICIENCES AND CHARACTERISTIC DAMAGES

3.1. Errors of structural design or/and inadequate later transformations

In practice, but also in some old technical references one can find solutions which prove the lack of structural understanding of the original constructor or early restaurateur. A significant example of incomplete understanding of the state of forces offers the solution presented in the Figure 10a [6]. There is no reason to raise the brick arch so much over the slab except the only situation: when the purpose is to let the possibility of creating holes in the wall later on. Leaving this last case aside, the best position of the discharging arch is that immediately over the slab like in Figure 8, which eliminates the shear forces in the vertical supporting wall between the two levels of supports. The suspender metal bars would be shorter and - the most important reason - the adjoining slabs can overtake the horizontal thrusts of the arch.

Another example is the old Baroque school building (Fig.11) at Orastie [2].

Figure 11

Figure 12

It was built at the end of the 18th century in masonry structure with brick vaults over the ground floor, brick groins in the corridor to the courtyard (Fig.12) and timber slab structure at the second level. At the end of the 19th century the school was extended and another block was adjoined in neoclassic style (that in the back side) to the first one (in the front side, Fig.11). In order to fit the exterior look of the older building to the façade of the later one series of new windows were cut in the front-side wall. In this way the corner zones of the elliptic brick vaults and supporting zones of the boundary arches were simply cut out (Figures 13 and 14) gravely disturbing the mechanical behaviour of the vault and supporting elements. Dangerous cracks have developed. Important horizontal cracks developed in the supporting wall (Fig.15) were caused by an original structural mistake [5]. Built in the Renaissance era [8] in the 16th century (from this time has remained the cylindrical vault covering the entrance passage) the building was transformed afterwards.

Figure 13

Figure 14

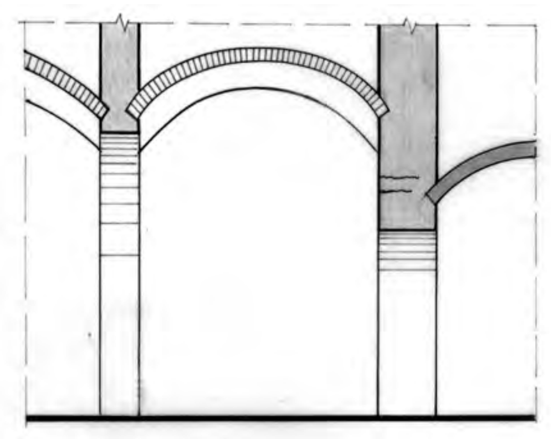


Figure 15

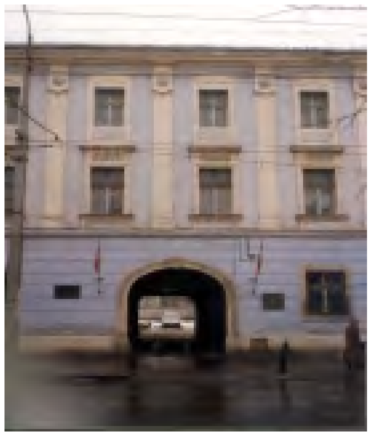


Figure 16

The elliptical vaults covering the ground floor were built in the Baroque epoch three centuries later. The supporting arches and points of these vaults were placed higher as the supporting line of the former built vault. From this reason the wall cracked between the two supporting levels in spite of its large thickness.

3.2. Foundation settlements

It is well known that foundation settlements, especially those differentiated or unequal in plan may cause important structural damages. This phenomena can be very dangerous in the case of masonry structures because of the lack of horizontal binding of the structure. Damages caused by foundations settlements may have various causes. The main causes: (a) error of design concerning the foundation depth related to the soil quality; (b) different settlements of certain parts erected at different times of the same building, or due to the partially in the plan developed basement; (c) flooding of the foundations.

The initial Renaissance-style building of the Ethnographic Museum of Cluj (Fig.16) was transformed in late-Baroque-Classicist style in the second half of the

19th century [8]. The Baroque vault of the entrance reached a state near the collapse due to the ceding of the foundations (Fig. 17 and 18).

Figure 17 Figure 18

The old school building in Orastie [2] seems to be built in two stages. The cracks roving through the whole height of the building (Fig.19) appeared at the joining of the two parts and are caused by the different settlements of them, being aggravated by a former (now abandoned) untight sewerage system and the damaged gutters.

Figure 19

3.3. The lack of horizontal binding

During practical investigations became obvious that the cracking of the brick vaults and arches, mainly in the middle of span, are caused by the lateral displacements of the supporting elements (walls). The mechanism of failure is well known (Fig.20).

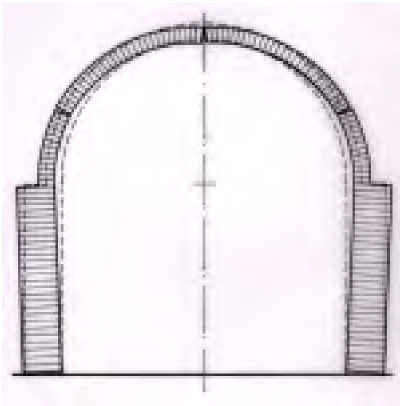


Figure 20 Figure 21

The visible cracks are usually those on the interior face in the middle of span (Fig.21). The insufficient rigidity of the marginal walls against the horizontal reactive forces can lead to lateral displacements and deformation of the wall (Fig.27) causing further cracking of the transverse supporting wall (Fig.22). Often serious damages are hidden by successive superficial repairs (Fig.23).

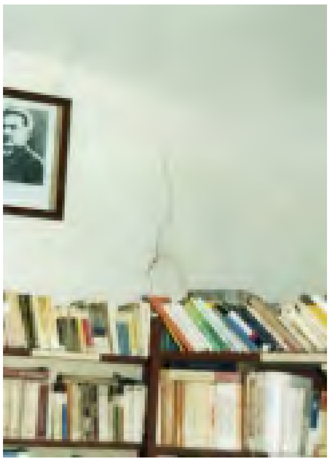


Figure 22 Figure 23 Figure 24

The influence of the external walls deformation and lateral displacement can be transmitted until the roof level and can be easily identified by the moved position of the roof timber elements and other signs (Fig.24).

4. STRENGHTENING SOLUTIONS FOR BRICK VAULT STRUCTURES

Any strengthening solution and operation has to take into account some principles concerning: (a) investigations on the structural damages, their type and gravity;

(b) architectural including both functional and esthetic criteria; (c) the compatibility between the original materials and that proposed for strengthening.

Our experience let us to conclude that many old masonry buildings are suffering because of the unequal settlements of foundations. The masonry vault structures are very sensitive on this damage due to the fact that they are conceived to work by compressive strength. In this order any significant deflection of the curved surface or arch points out the compressive force relative to the pressure line leading to bending forces. The section being not able to take over tensile stresses the cracking is imminent. In the very case of our school building [2] a joining girder, actually a deep perimeter beam of reinforced concrete as well as strengthening foundation blocks were provided (Fig.25). This solution was also justified by a very bad technical state of the elevation wall (Fig.26).

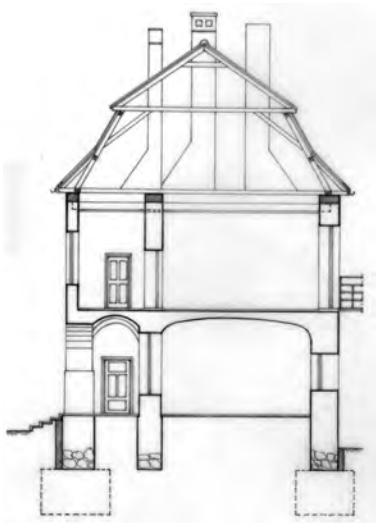


Figure 25

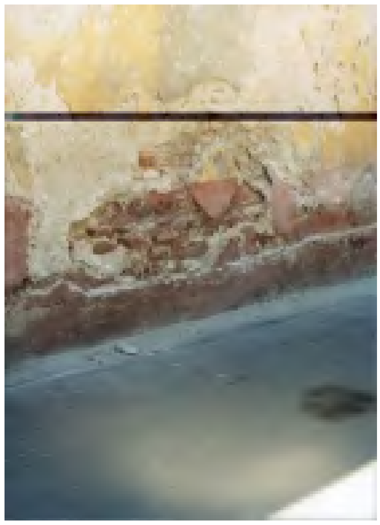


Figure 26

The characteristic strengthening solutions for brick vaults are those refering to characteristic damages due to the lack of horizontal binding elements. Two main problems have to be resolved: to overtake the horizontal reactive forces of the vaults and to restore the brick vault. There are many factors which should influence the choise of the proper solution. The horizontal thrusts can be overtaken by tie-bars of several types or in several positions according to the structural form, height of supports, accessibility, esthetical considerations etc. (Fig.27 - solution *b* or *c* and Fig.28 - solution *a, b* or *c*). In our very case the plan of placing metal tie-bars is presented (Fig.29). It is provided to put the tie-bars under initially tension. Concerning the vault restoring the potential collapse mechanism and the actual cracks pattern are conclusive on the mode of intervention. Taking into account the former presented collapse mechanism (Fig.20) the ideal solution for restoring the vault would be that of completing the wall with a tensile resistant (reinforced) and also compression resistant repairing material on the both sides of the vault, intrados and extrados. In practice the vault strengthening occur usually by applying only an upper or only a lower complementary reinforced mortar layer to the existing vault. An extrados-only restoration technique for brick vaults was presented recently by M. Sassu [9].

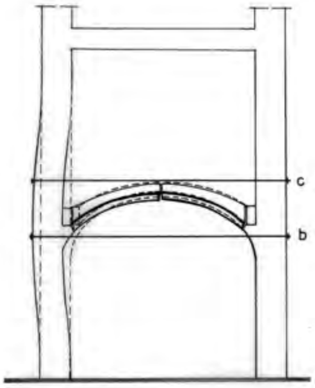


Figure 27

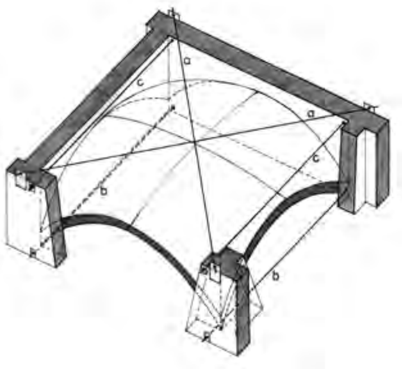


Figure 28

This solution was chosen from several reasons (intrados bearing artistic decorative elements, vault inaccessible at its intrados etc.).

Other times intrados-only solutions are applied, like in Fig.30 [2], on the boundary arches. The supporting masonry pillars were also strengthened by a reinforced mortar layer with a thickness of 10 cm.

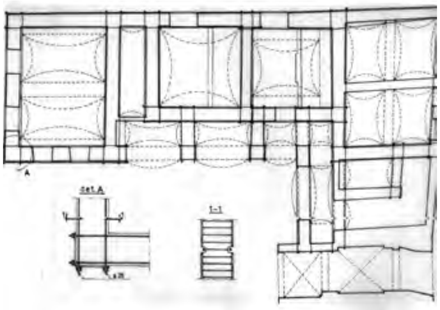


Figure 29

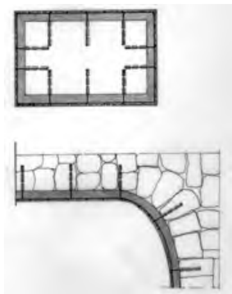


Figure 30

5. CONCLUSIONS

It is to be mentioned that, in our opinion, in spite of absolutely true and reasonable worries concerning the proper behaviour of the brick masonry strengthened by reinforced concrete or mortar there are many cases when this or similar techniques are the only available. It doesn't means that the problem of physical and chemical incompatibilities or difficulties should be neglected. A very careful examination of the materials is necessary in each case in order to find the proper solutions and technologies. Beside the traditional reinforced mortar or ferrocement techniques using special cements the today's researching are directed to new solutions which may assure an intimate connection between the old and new material, may avoid the water percolating through, or altering the degree of humidity within the brick and mortar structure by using special reinforcement meshes and mortar components or other filling materials. But this is another field of the same researching.

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HAGIA SOPHIA: GEOMETRY AND COLLAPSE MECHANISMS

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ABSTRACT

Since its erection in the Byzantine Age the Hagia Sophia Dome has collapsed three times due to earthquakes. After restorations and reinforcing had been carried out by the Ottomans and particularly by Sinan in the 16th century as well as later in the 19th, the main displacements stopped and not even recent severe earthquakes have succeeded in damaging it.

The collapse mechanisms comprehension as well as the successful strengthening of the entire structure, performed during the centuries, provides basic indications about the behaviour and the deficiency of the structural system, which are not only useful to Hagia Sophia future safeguard, but also to understand the seismic behaviour of such a kind of big stone buildings and the criteria to be adopted for their strengthening.

1. INTRODUCTION

No introduction is needed for Hagia Sophia - one of the most important and wonderful outcome of human talent and creativity - studies on it are so many and varied that it is almost impossible for anyone to add anything new to them.

Nevertheless the complexity of its building is such that any scholar who concentrates his/her interest in it is always spurred towards further research work. Hagia Sophia is not only an exceptional architectonic masterpiece capable of constant revelations about its shape and building techniques, but far more it is a dynamic reality, which more than any other great monument in the history of architecture has undergone continuous modifications and restorations.

The present building, we can see now, is the result of failures, reconstruction, restorations, strengthening, enlargements, transformations and deprivations: built as a church, it became a mosque and it is now a museum.

Each modification, alteration and transformation was done following specific needs and in compliance with precise rules, and for that reason it is historically and scientifically interesting.

Besides the most immediately perceivable historical, stylistic and architectural aspects, the behaviour of its structures and materials during the years, the building and consolidating techniques are of great interest in Hagia Sophia mostly in the light of the several earthquakes it has endured.

The very many failures and consequent restoration as well as the results of such interventions bring about a further motivation for studying Hagia Sophia which can be considered a historical restoration and the book which clearly reports the signs of the damage suffered and the results of the restoration performed.

Hagia Sophia's history is closely tied to Istanbul seismic history. Few years after its building, its dome entirely collapsed due to an earthquake and, during the 10^{th} and 14^{th} century, the dome suffered two further partial collapses. Restorations and structure reinforcements followed each collapse or failure. To understand the causes and the effects of the interventions performed as well as the modifications made, it is therefore necessary to know how failures and displacements occurred. To evaluate the size of the displacements and to understand the mechanisms which generate them, it is necessary to know the original building structure. The above procedures – that is, the survey of displacements of the existing building for an understanding of the collapse mechanisms and consequently of the intrinsic weakness of a structure – is in actual fact, the “empiric-experimental” method architects followed for centuries, until the 19^{th} century, to learn how to build correctly.

Owing to the introduction of new building materials such as steel and reinforced concrete, in the second half of the 19^{th} century, building could no longer be referred to preceding experiences and a new science, capable of “foreseeing” the behaviours of new buildings through structural calculation, had to be invented.

For over a hundred years architects forgot both traditional rules and empirical methods of structural analysis which had allowed the building of wonderful works for centuries even in seismic areas.

Nowadays when the historical architectural heritage is regarded by all peoples as a fundamental cultural wealth to be safeguarded, we have had to face the fact that the “calculation” procedures to foresee the structural behaviour of modern buildings in iron and concrete are not at all capable of giving adequate indications in the field of ancient masonry structures, mostly for seismic prevention. The complexity of ancient buildings, lack of homogeneity of the materials, un-linearity in structural behaviour, uncertain response to seismic events, alteration over the years, unpredictable historical events make accurate analysis through non-linear procedures based on the use of finite elements, highly fanciful and often misleading.

This awareness has created a new interest in ancient empirical procedures in displacement survey and structural behaviour analysis, which have been revised in the light of the most recently acquired knowledge. Empirical analysis, in particular, has been associated with the latest collapse analysis, thus creating a new static and seismic analysis procedure which could be termed "collapse survey through collapse empirical analysis". The study of elastic ranges is given up in order to directly examine collapse situations.

The application of this method to such a complex building as Hagia Sophia required a preliminary study of the original geometric shapes, this to single out the movements which occurred over the years and the mechanisms responsible for the different collapses.

Finding these mechanisms helped understand past reinforcements and the reasons which have led to the positive up-to-date behaviour of the great building. This operation was made easier by the fact that Hagia Sophia – in spite of collapses – was built according to a strict, although complex, geometry.

It was not by chance that Justinian had entrusted two scholars: Isidoro and Anthemio di Tralle with the building of the new great church. Both were well-known for their studies of geometry and mechanics.

The new precise survey carried out by the Japanese group led by Prof. Hidaka has made it possible to reconstruct, in a reliable way, the original geometry and therefore to understand and evaluate, better than ever before, the displacements occurred throughout the centuries.

2. GEOMETRY AND DISPLACEMENTS

2.1. Ideal geometric reference

A precise survey of the building geometry is the basic step for any research; that is more veritable for such complex monument as Hagia Sophia and, above all, if a study of the displacements occurred during the centuries.

Particularly important is the comparison between the present geometry and the one which can be reasonably hypothised as undeformed.

The notes that will be later reported, on Hagia Sophia's geometry, are finalised to an understanding of those deformations occurred over the centuries by hypotesising as initial geometry the one illustrated in the already mentioned paper by Blasi and Bianchini. Robert Van Nice [1] has the very great merit of having supplied us with an exceptional survey of Hagia Sophia; the result of a life wholly devoted to this monumental survey work.

All later scholars have referred to Van Nice's survey.

Nowadays, 40 years later such survey, the Japanese team directed by Prof. Kenichiro Hidaka has done an instrumental survey of the Hagia Sophia basic sections. Thanks to previous surveys, it was already known that the dome had been erected on a square with a 100 feet side base.

Mainstone found very interesting proportions and relationships in the base geometry.

It could not be otherwise, due to the architectural characteristic of that age and the references to Roman architecture. It is unthinkable that such a large building could be built without a strict observance of proportions and building rules, taking into account that the two architects, Isidoro and Anthemio were famous for their geometrical and mechanical knowledge. The complete identification of the base geometry is still difficult if the volumes in elevation are not considered or if the building three-dimensional geometry, is not found out. The analysis of the three-dimensional geometry is still a very complicated operation due to the fact that remarkable displacements and reconstruction have cancelled the original geometry and any hypothesis is made quite difficult.

Van Nice had supposed in his drawings that the pendentives had a 75 feet bend radius, but did not make further hypothesis about the whole distribution of volume.

Actually recent surveys have confirmed that, if pillars deformation is left out, it is possible to inscribe a sphere with 150 feet diameter inside the whole volume restricted between the pendentives and the floor (Figure 1). Therefore the centre of the whole building was established at the a height of 75 feet. When comparing Hagia Sophia to Pantheon you can see an evident co-relation between the two buildings: Hagia Sophia was practically built with a volumetry corresponding to that of Pantheon, equally built around a 150 feet diameter sphere.

Thanks to the more recent dome, built over pendentives, Hagia Sophia's internal height was 156 feet, i.e. 6 feet higher than Pantheon. Today with its higher dome re-built in 557-562 Hagia Sophia's internal height is about 176 feet.
Surely to Giustiniano, the building of a "new temple" for the "new religion" of the Empire "new capital" which had dimensions and volume corresponding to those of Pantheon but with a higher dome, must have had a particular political religious meaning.

Simularly a thousand years later Sinan must have worked at this mosques.

Figure 1. Axonometric projection – internal volumes – of the Hagia Sophia's structure (drawing by L. Bianchini).

2.2. Hypothesis about the original dome shape

Several hypotheses have been made as regard the original dome geometry, but it appears that probably in spite of the deformations in the pillars occurred during building, this dome was erected on a base perimeter which had a precise geometric shape. The surveys performed by the Japanese team demonstrated that the marks on the gallery stones at the dome base, coincided exactly, before deformations, to arches with a 50 feet radius circumference [2].

Very likely those marks were the reference marks for the dome layout. It is not possible to ascertain if these arches were sections of a sole circumference or if they were parts of a geometric form having four centres, as Mainstone supposed.

As the pendentive shapes appear remarkably corrected, we like to think that the ancient builders succeeded in erecting the first dome on a perfectly circular base, 50 feet in diameter certainly corresponding to the shape which had been thought of when projecting. In any case, the tracks of arches whose circumference is 50 feet confirm that the building was erected in compliance with precise geometry notwithstanding the deformations occurred during construction. Therefore it can be assumed that even the dome was erected according to the monument comprehensive geometry. Hence the dome may have had the same centre as the sphere with 150 feet diameter.

Considering this hypothesis and that it is commonly accepted that the first dome was 20 feet lower than the present one, it is easy to draw it: its radius is 6 feet longer than that of the sphere 150 feet in diameter.

The first dome had a 81 feet radius. The present one has as 54 feet radius. Therefore it has a remarkably wider curvature than the original one. This is a fundamental factor to understand the sequence of collapses undergone by the monument. The hypothesis shown is very close to Antoniades [3] and agrees with description of Procopius, who gave an account of the amazement in front of a spherical dome illuminated by the light of the windows (Figure 3,4,5 and 1).

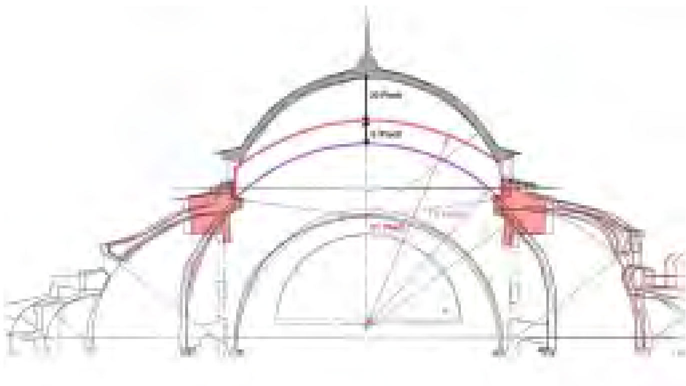

Figure 2. R. Van Nice: cross section. It shows the pillars, deformed and rectified during the erection, and the probable shape of the original dome.

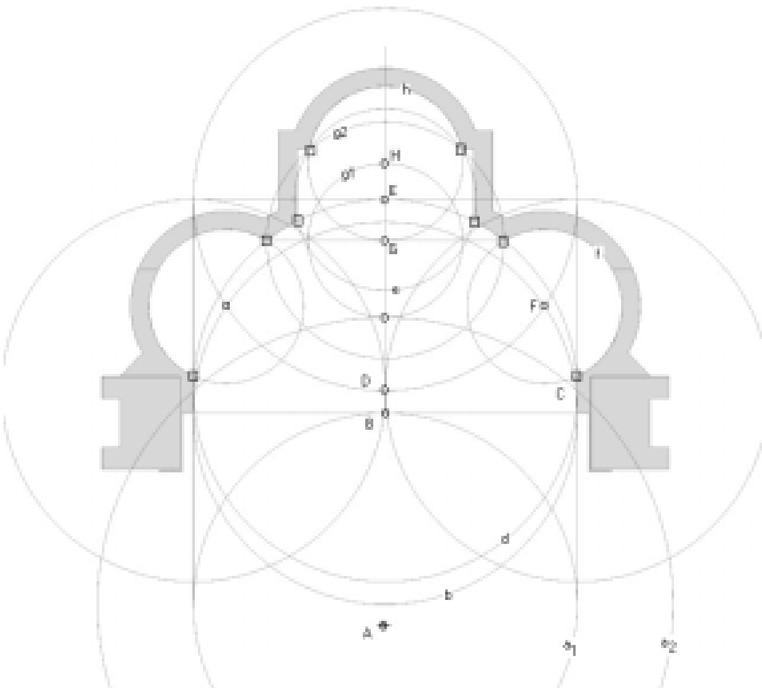

Figure 3. Hagia Sophia: ground floor plan tracing by means of circumferences (following the alphabetic order of the letters).

Figure 4. Hagia Sophia: complete volume of the central nave.

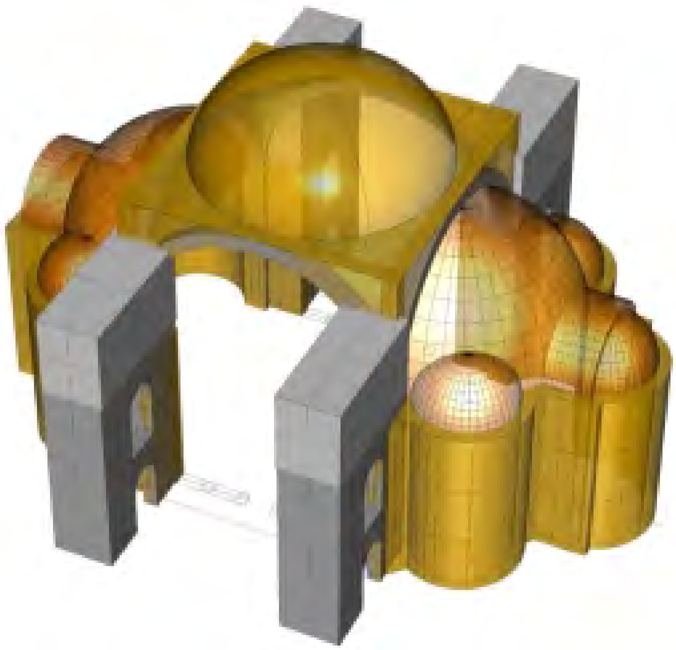

Figure 5. Hagia Sophia: axonometric projection of the present situation.

2.3. Hypothesis on the original shape of the square drum

Today the dome rises, outwardly, over a drum which has a plan form almost square in shape. Probably the original form was different and that is clearly evident if the present displaced and rotated position of all the stairs placed at the 4 corners is examined.

If we analyse Van Nice's drawings, which accurately show the masonry used to rectify the walls, we can locate the stairs in their original place. Relocating the stairs at the original position, parallel to the building axes, it is obvious that the external wall surfaces of the stairs were laid remarkably behind compared to the present external surface of the outer arches (Figure 6). The situation of the four corners of the square drum is similar.

It is not reasonable to think that the stairs were located behind the outer arches otherwise they would have been without supports.

Is therefore evident that the main outer arches were built, in their present shape, after the stairs when the latter were rectified so as to have the same alignment as the fronts. The main outer arches probably were much less deep than they are now. Probably the same way as the nearby Hagia Irene.

As there are no news about the restorations of the outer arches during the 9th and the 14th centuries, therefore the translation and rotation of the stairs must have taken place partly during the building and partly after collapses in 557.

Figure 6. Stairs to the terrace on the square drum. In the masonry they are turned and displaced. Repositioning the stairs in their original position, it come out clearly that their external surface was not aligned to the present external one of the main arches, on the Northern and Southern sides. It is therefore probable that the external arches have been substantially modified and enlarged once the deformation occurred, due to the first collapse, in the 6th century, when the external stair surface was rectified.

3. COLLAPSE MECHANISMS

As already mentioned, the major cause of the Hagia Sophia heavy failures is the poor resistance of the transversal pillars, which have to bear the thrust of the dome and of the main internal arches. Such a deficiency appears particularly during seismic events. But to understand the reasons of the sequence of the collapses (firstly the entire dome collapsed, then, in the 10^{th} and 14^{th} centuries, there were two partial symmetrical collapses) it is necessary to understand which were the collapse mechanisms leading to situations of fracture.

The analysis of the original geometry shows that the first dome was remarkably lower than any other structural element.

The dome was the first element to collapse, due to the deformations of the pillars originated by the static thrusts of the structures as well as seismic actions, in accordance with the usual mechanism of domes, as it is shown in Figure 7 [4].
The new dome, rebuilt with a remarkably higher curvature, appeared more stable than the former one.

The weaker elements, with smaller curvature have therefore become the internal transversal main arches, surely already ruined by the deformation of the pillars.

The following two collapses (identical even if occurred 4 centuries apart) are due to the sinking of the internal transversal main arches and the mechanisms that produced them are the arch classical one (Figure 8).
In both cases, the collapse of the main arches involved a section of the dome that resisted on the whole.

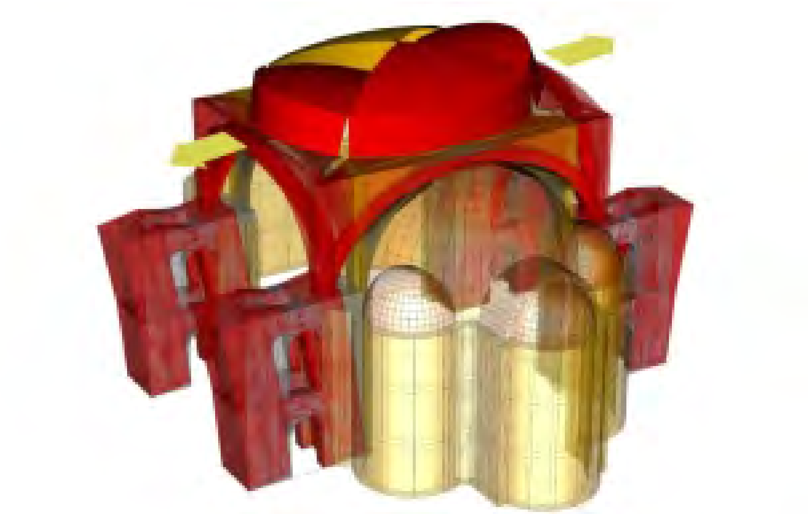

Figure 7. Dome collapse mechanism in the 6^{th} century
(drawing by L. Bianchini).

Figure 8. Collapse mechanism of the transversal main arches in the 10th and 14th century (drawing by L. Bianchini).

4. THE MAIN ARCHES CONTRASTING THE DOME

At the end of the 19th century Hagia Sophia was provided with four main arches, taken away by Fossati during the erection of the new hoops both in the dome and the drum.

Surely Fossati removed them because he thought that by hooping the dome at its base, they were no longer necessary.

But today by examining the dome ribs where in the windows are, it is possible to notice a widely spread group of lesions present in a regular, repetitive way all over the ribs (Figure 9 and 10).

They are slightly bent lesions in the intradox of the ribs, which reveal the presence of excessive outward eccentricity of the load. These lesions certainly appeared after Fossati's restoration and report a damage mechanism due to the absence of contrast at a higher level that of the hoop.

An accurate investigation of the wall decoration near the windows reveals, in the upper part, how a hoop, probably made of wood, was removed.

The main arches as well as the hoops on the windows were a valid opposition to the dome thrusts when lead at correct level. Hence the removal was probably not a wise decision.

Such main arches are always present in the great mosques of the Ottomans.

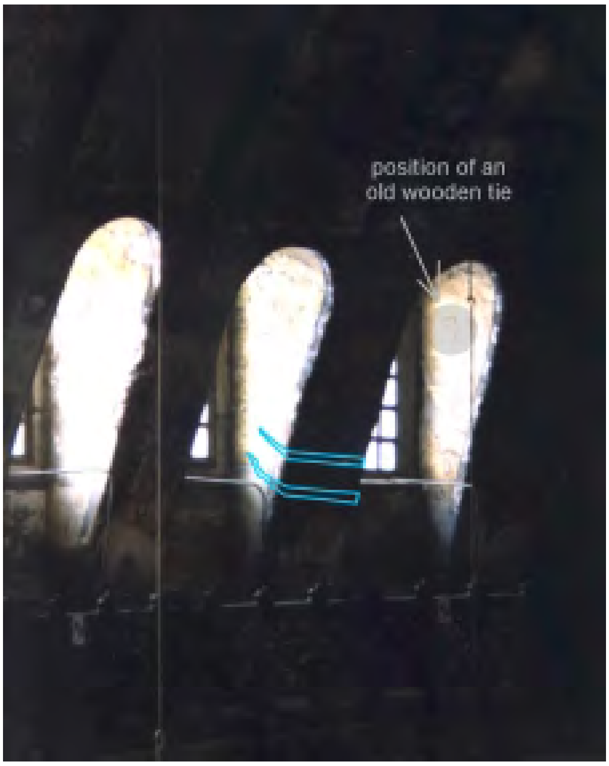

Figure 9. Photo of the windows at the base of the dome. The cracks on the ribs and the position of the old wooden ties are clearly shown.

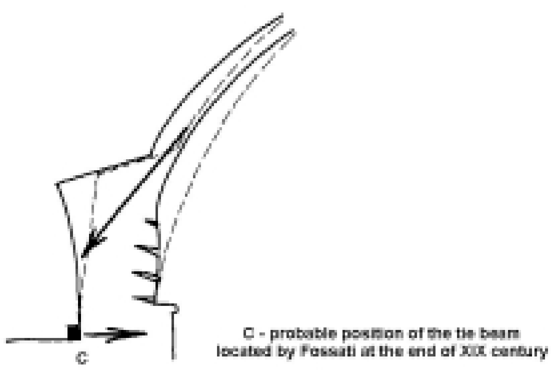

Figure 10. Scheme of the lesions in the ribs at the base of the dome.

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STRUCTURAL ANALYSIS OF THE PHASES OF CONSTRUCTION: DISCOVERING THE SECRETS OF THE ANCIENT MASTERS

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ABSTRACT

In order to be able to resolve problems of maintenance and restoration of architectural monuments, it is important to understand their structural logic. This includes also comprehending the methods of designing and building of their architects, i.e. their way of thinking and resolving architectural and technical problems. In order to get familiar with the ancient masters' working methods, I made the historical and structural analysis of the construction of the Šibenik cathedral (a masterpiece of the Renaissance architecture in Croatia). Research of the historical evidence enabled to establish the phases of the construction, which I then analysed, using a modern computational programme for masonry structures. Thus, virtual experiments granted an insight into some of the ancient masters' secrets.

1. INTRODUCTION

1.1. Objective

The objective of the structural analysis of the Šibenik cathedral in course of its construction is to understand the ancient masters' method of designing and building. The architects and builders of the past, although not in possession of our modern methods of mechanical analysis, created buildings of outstanding beauty, structural logic and boldness. For example, the architects of the Šibenik cathedral constructed it in Quattrocento and completed it in Cinquecento, when not even the physical concept of mechanical forces was developed. In spite of this, they created an audacious structure, unique in its method of construction and in the resulting mechanical behaviour. Their knowledge of the mechanical characteristics of the structure was based on daily observing its behaviour, its microfissure pattern, and as soon as they noted any problem, they intervened. [1] The achitects of the Šibenik cathedral were not in position to learn from earlier buildings (as ancient master builders usually did, using existing buildings of the same type as real-scale models) [6], since there were no predecessors of this type of construction. [10]

1.2. Method

With this objective in mind, it was necessary to establish the phases of construction, in order to analyse the structure as it actually had been in each phase, and as its architects could have observed it. Therefore, a comprehensive historical analysis of the building process was necessary, based on the historical evidence, such as the documents of the *fabrica* [2], the coats of arms carved in the stone of the cathedral, and the contemporary chronicles. The logic of construction was also taken into consideration, as well as our interactive knowledge based on the mechanical analysis. Since masonry displays mechanical behaviour rather different from that of standard modern structures [1], for the analysis of the established building phases I used CALPA – a modern computational programme for masonry structures [7]. Thus, virtual experiments made it possible to establish the role of each structural part, to understand the logic of construction and structure, and to discover some of the secrets of the ancient masters.

2. THE UNIQUE STRUCTURE OF THE ŠIBENIK CATHEDRAL

The Šibenik cathedral is a masterpiece of the Croatian Quattrocento architecture. It excels in beauty, which results from its unique stone structure. The apsidal part of the cathedral and its upper vaults (i.e. vaults of the nave, aisles, transept and presbytery, as well as its dome) were constructed by using a special technique of assembling large thin monolithic elements, wedged into slender stone frames. The monolithic slabs of the vaults, spanning 3.0-4.2 m, are only 15-25 cm thick (their thickness varies, since they are shaped as roof tiles, with overlapping). The arches of the barrel vault of the nave, presbytery and transept span cca 7.75 m, and their cross section is cca 80 cm wide and cca 55 cm high. The arches are tightened with iron tie-rods, which enables the slender substructure, without any buttressing system. [9]

Only recently, after the war damage in 1991, the experts were in position to research the details of this supreme technique of assembling [11], and were astonished by the inventivness, skill and precision of its builders. The assembling technique, unique among stone constructions, seems to be akin to timber constructions. High-quality limestone from the island of Brač, Dalmatia (with its compressive strength σ_c = -118.5 MPa, and after a cycle of freezing its compressive strength σ_{cf} = -74.5 MPa), [9] and very precise carving made it possible that the thin vaults form also the covering of the church. Thus, the external form corresponds entirely with the inner space, which is a modern architectural concept, very rarely to be found before the 20th century. The unique structure and the white stone, shining under Mediterranean sun, make it a jewel of Renaissance architecture, which was recently added to the UNESCO World Heritage List.

3. PHASES OF CONSTRUCTION

3.1. The first phase (1431-1441)

Documents from the time of construction prove that in 1402 the citizens of Šibenik decided to build a new cathedral, larger and more splendid than the older one had been. Nevertheless, the construction began only in 1431 [2], probably after the design of *protomagister Boninus de Milano* (Bonino of Milano), who had died in 1429, i.e. before the construction was started. [8] In the first year (1431) the *fabrica* was directed by *protomagister Francuscus quandam Jacobi de Venetiis,* and later on by two minor Venetian masters: Antonio Busato and Lorenzo Pincino. [5] They began to build the western part of the church in Lombard Gothic manner, as simple three-aisled basilica, with massive stone walls and small openings, and with Gothic ribbed groin vaults spanning the aisles. [2]

3.2. Construction under *protomagister Georgius Matthei Dalmaticus*

After certain "errors and omissions" in the construction had been observed, in 1441 a new protomagister was invited from Venice: *Georgius Matthei,* native of Zadar (Croatia), who called himself *Dalmaticus*. Master George obviously had built himself a reputation and fame in Venice. This is the only plausible explanation of his using the name "The Dalmatian" while working in the very heart of Dalmatia.

He changed the previous plan of the cathedral, extending it in the eastern direction, and introducing a transept with crossing, which he obviously intended to be crowned by a dome. [2] He also changed the method of building: instead of using customary massive masonry, he constructed apses consisting of large thin stone slabs wedged into stone frames. [4] Historic evidence and art-historic analysis prove, that it was him who completed the aisles up to the groin vault, since most column capitals display his style. He also built the aisle groin vault, but he was obliged by the contract to construct it after the model of the ribbed groin vault of the first bay, which had been constructed earlier. He completed the nave wall up to the frieze of leaves "swirled by the wind", [2] and the vigorous columns of the crossing, up to the same height.

3.3. Construction under *protomagister Nicolaus Joannis Florentinus*

The wall of the nave had been completed up to the double frieze of leaves in the Dalmatian's period, and the openings of the triforium, in the form of Doric frieze of "missing metopes" between *pilastrini* in the form of "triglyphs", were designed by Niccolò Fiorentino *(Nicolaus Joannis Florentinus),* who assumed the position of protomagister in 1477. [2] This artist, educated in Florence, accepted the Dalmatian's constructing method [3]: he completed the eastern part of the church, constructed vaults of the transept and the dome, and began to build the nave barrel vault, consistently using the technique based on the Dalmatian's idea of assembling large stone slabs into the frames (which, in the case of vaults, are slender stone arches). The dome was completed by 1499 [2].

3.4. Completion of the construction in Cinquecento

After the death of the master Niccolò in 1505, the cathedral was completed after his design by two *protomagistri* from Mestre (near Venice): *Bartolomeus Jacobi de Mestre* (*protomagister* from 1517) and his son *Jacobus* (in charge after 1526). They constructed the barrel vaults of the nave and semi-barrel vaults of the aisles. In 1536 the church was completed by placing the "key-slab" of the first bay of the nave vault, and simultaneously, the keystone of the western façade. [2]

Figure 1. The unique vaults of the cathedral in Šibenik (Croatia)

4. STRUCTURAL ANALYSIS OF THE CONSTRUCTION PHASES

4.1. Construction phases and virtual experiment

As it can be understood from the above exposed, the phases of building differ in their artistic style, and, more important, in the method of construction. Therefore, I believe that the established phases of building can be accepted as sufficiently accurate.

It is very interesting to investigate how these different construction methods and varying thickness of elements affect the mechanical behaviour of the building.

But, above all, I tried to understand how the structure behaved while it was under construction, and how its behaviour changed as it grew. Which events in the construction were crucial, or critical, changing its behaviour radically, and why? How the critical zones changed as the construction progressed? And how the architects coped with the problems? Can we recognize some critical points, which were recognized as such by the master-builders themselves, and understand emergency measures they took to prevent the damage?

Also, since there are "experts" who consider the iron ties of the Šibenik cathedral "ugly botching, to be cut off as soon as possible", I paid special attention to the research of the structural role of the iron ties, to be found in all the vaults of the church.

In search of answers to all these questions, I made virtual experiments – simulating the structure in certain phases of construction, as well as the structure with and without tie-rods.

4.2. Construction completed by George the Dalmatian

I first analysed the state of the structure as had been completed under the direction of George the Dalmatian: the presbytery almost finished, the groin vaults of the aisles completed, and the nave wall built up to the frieze of leaves. The structure of the nave and aisles was determined by the Dalmatian's predecessors: the spans, diameter of columns, thickness of the walls, even the shape of the ribbed groin vaults of the aisles. [2] Thus, one can not consider it a Dalmatian's design.

In a virtual experiment, I simulated the construction in this phase without tie-rods and found out that, surprisingly, even in this early phase of construction the ties were necessary. Without the ties of the transverse ribs, the horizontal thrust of transverse arches of the groin vault would have caused horizontal displacements, which, not withstood by the load of the upper parts of the building (not constructed by then), would have caused cracks. The continuous fractured zone in the middle of the diaphragm and in the transverse rib would have alarmed master-builders to take measures to prevent further damage. Indeed, were those cracks the reason why the Counsel of Šibenik invited a new *protomagister*?

Figure 2. Principal compressive stresses - structure completed by master George

From a historical document, we know that in 1452 two *catenas ferreas largas et laboratas pro archivolto* were made for the *fabrica*. [2] Both this document and the mechanical analysis make us quite sure that the iron ties are an integral part of the structure, built in simultaneously with the erection of groin vaults.

In the virtual structure observed, the active structure was “split” into two independent parts, with eccentricity of the axial force in the column, causing compressive stress as high as –5.82 MPa in this early stage of construction. Of course, it is still much lower than the compressive strength of the white limestone, used for the construction of the cathedral: -74.5 MPa (after a cycle of freezing).[9]

4.3. Construction completed up to the upper semi-barrel vaults of the aisles

The next stage of the construction I analysed, was the structure with completed semi-barrel upper vault of the aisles. In order to prove the crucial structural role of tie-rods, I analysed it both with and without them.

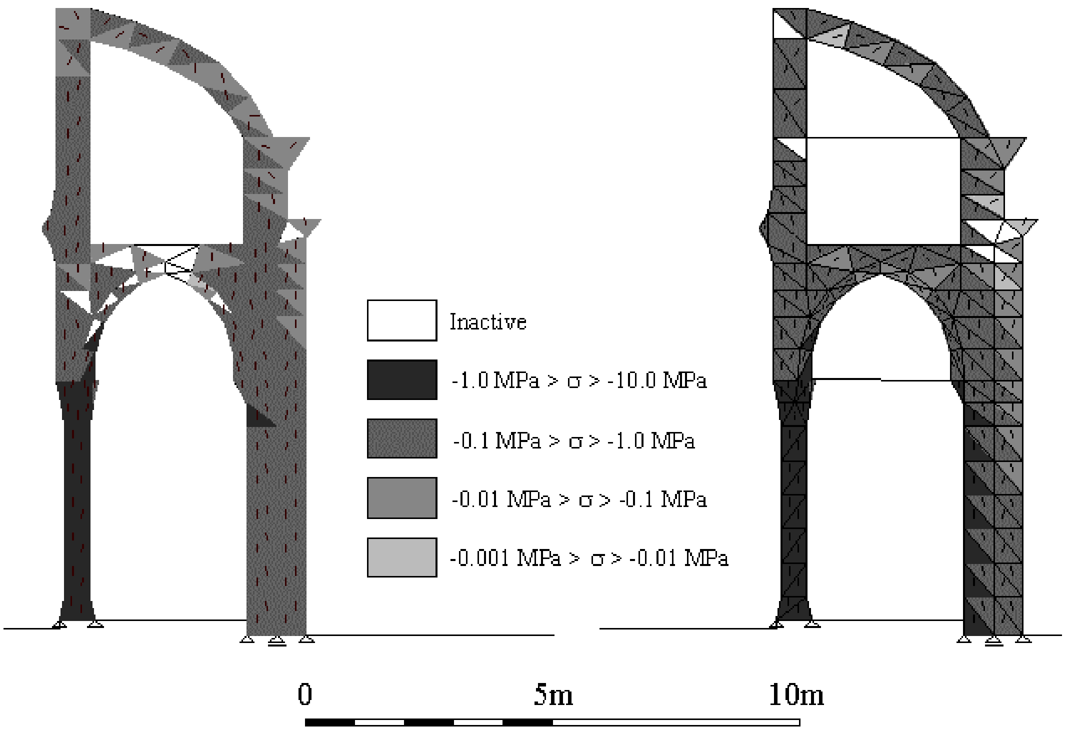

Figure 3. Prinicipal compressive stresses in the structure completed up to the upper semi-barrel vaults of the aisles
a) without tie-rods (virtual structure) - left b) with iron ties (real structure) - right

4.3.1. Virtual structure, completed to the semi-barrel aisle vaults, without ties

The virtual structure completed up to the upper semi-barrel vaults of the aisles, when analysed without the ties, shows similar weaknesses as does the previous phase of the building without ties (Case 4.2.). The continuous critical zone in the middle of the diaphragm, in the extrados of the transverse rib, as well as an inactivated element in the nave pilaster and in the façade pilaster near the lower façade cornice, show a mechanical behaviour very similar to the one of the structure in the previous phase (Case 4.2.). Continuous cracked zone in the transverse rib and in its diaphragm would have probably caused a collapse of the groin vault.

The highest compressive stresses (up to –7.41 MPa) appear in the column shaft, due to the eccentricity of the axial force.

4.3.2. Construction completed up to the semi-barrel aisle vaults, with ties

On the other hand, the structure in the same phase, but provided with iron tie-rods both for the lower, groin vaults and for the upper, semi-barrel vaults, shows completely different pattern of stresses and strains. There is no critical zone in the groin vault and its diaphragm. The structure is perfectly stable in its lower part, and even the stresses in the column are very evenly distributed, not exceeding –4.77 MPa in the column shaft. The even distribution of stresses is very important for the column - a slender structural element, in which heavy load is concentrated.

This structure shows other weak points. One of them is a support of the semi-barrel vault arch on the top of the nave pilaster, due to the displacement caused by the horizontal thrust of the arch, not withstood by any force at this point in this stage of construction.

Another one is the anchoring point of the tie-rod on the same pilaster, due to the concentrated force of the tie-rod. The third critical zone is a façade pilaster at the height of the lower cornice. It is caused by the change of the cross section of the structural element, in which the outer part is not loaded at all, while the inner part is carrying the heavy load of the vault and of the wall above it. Nevertheless, in spite of several weak points, the structure with ties is obviously far more stable and favourable than the one without them.

4.4. Completed structure

Although it is obvious that the audacious and slender structure of the Šibenik cathedral required tie-rods from the very beginning of the construction of vaults, I have done the analysis of the completed structure both with and without ties.

It is a virtual experiment, which enables us to understand the structural role of the iron ties. Virtual experiments with simulated structures, feasible due to computers, enable us to analyse mechanical functions of different structural parts of buildings, and to research and compare different types of structures. Thus the structural "virtual experiments" enable us to understand essential characteristics of mechanical behaviour of ancient buildings and to regain the lost knowledge of the past generations of great masters.

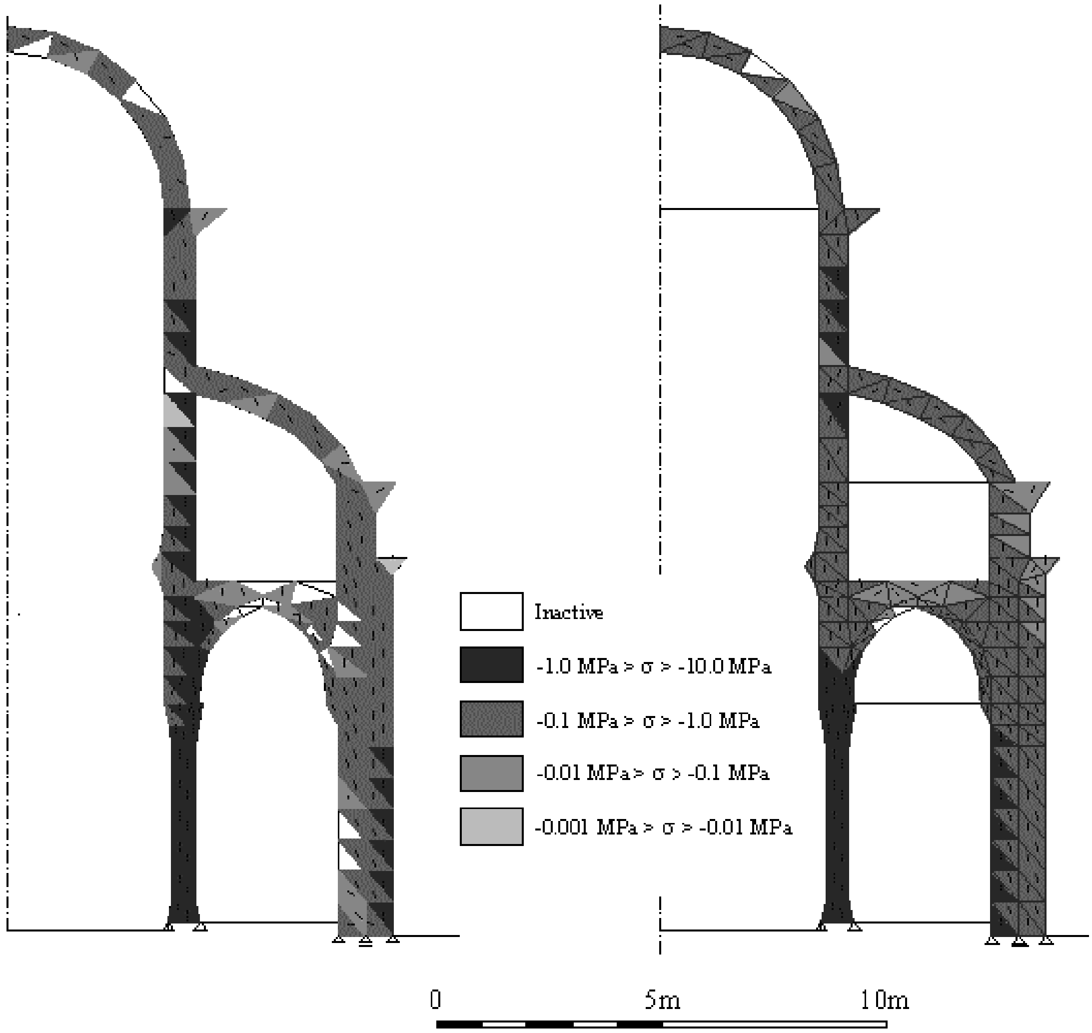

Figure 4. Prinicipal compressive stresses in the completed structure
a) without tie-rods (virtual structure) - left b) with iron ties (real structure) - right

4.4.1. Virtual completed structure without tie-rods

The completed structure without ties shows considerable weaknesses. It has some critical zones, the most dangerous being the diminished active cross section of the transverse arch of the nave barrel vault at three points of the arch: one in the middle of its span and two on its sides. This pattern could eventually lead to the collapse of the barrel vault.

On the other hand, the continuous inactivated zone in the transverse rib of the groin vault and its diaphragm is equally hazardous. The support of the semi-barrel aisle vault on the nave pilaster is also a critical point.

In the completed structure without ties yet another critical zone appears: in the façade pilaster, in its lower part, near its inner surface. It is caused by the eccentricity of the axial force in the pilaster, due to the thrust of the vaults. Thus, the compressive stress of –3.04 MPa appears near the outer surface of the lower part of the façade pilaster, while maximal stress in the nave column (-6.58 MPa) appears in its upper part.

4.4.2. Completed structure with tie-rods – Real structure

On the contrary, the completed structure provided with tie-rods displays a completely different mechanical behaviour. Critical zones and points are reduced to a minimum: only three inactivated elements are the absolute minimum among the analysed structural phases.

Two critical points, one on each side of the arch of the nave barrel vault, due to the deviation of the form of the arch axis (semicircle) from the pressure line (a parabola, for this state of action) can not jeopardize the structure.

Other two inactivated elements are on the intrados of the transverse rib of the groin vaults, and are not important at all, since they are marginal elements of a large solid structural zone.

To sustain the claim of a mechanically favourable structure, I should mention that the compressive force in the column is centric, the stresses in it not exceeding –5.88 MPa. This is the highest compressive stress in this structure, obviously much lower than the compressive strength of the stone used (σ_{cf} = -74.5 MPa).

5. CONCLUSIONS

The structural analysis and virtual experiments with the unique structure of the Šibenik cathedral, which was recently added to the UNESCO World Heritage List, enabled us to better understand its mechanical behaviour, as well as the role of certain structural elements and their structural logic and necessity.

Computational analysis of the construction of the Šibenik cathedral, by using CALPA programme, has displayed the behaviour of the structure in several phases of building, established from historical evidence. The comparison of the mechanical characteristics, and the state of stresses and strains in the analysed phases, enables us to observe the cathedral through the eyes of its builders.

The comparison of virtual structures with and without the tie-rods proves that the ties were necessary from the very beginning of the construction of the vaults. Together with the historic evidence indicating that the ties were implemented throughout the construction, it removes all doubt that the iron ties are an essential and indispensable element of this structure.

Thus, the structural analysis can resolve several art-historians' dilemmas on the construction of a building. Structural logic and exact mechanical research can verify whether certain hypotheses are plausible from the structural point of view.

It is absolutely necessary to understand the structure, its behaviour and the real function of its elements, especially when we are faced with the need to repair and strengthen a damaged or endangered structure. Without this knowledge, the restorers may not only diminish the artistic value of the monument (and for architectural monument, a structure is undoubtedly a crucial component of its artistic value), but they may also harm its structural system and jeopardize its stability.

By understanding the ancient architects' methods of design and construction we will eventually become able to regain their “structural intuition” (which in fact is the ability to create logical links between many data). The computer is a great aid to reestablish the “forgotten knowledge” of the great master-builders of the past.

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SAFETY ASSESSMENT OF ANCIENT MASONRY TOWERS

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ABSTRACT

The results of structural analyses, carried out to assess the safety of a medieval masonry tower (the belltower of the Cathedral of Monza, Italy), are presented. The analyses were performed by means of a commercial finite element code with user-oriented interface. The interface was utilized to incorporate in the code a theoretical model, expressly developed to describe the creep behaviour of masonry up to failure, accounting for the evolution of damage. The reliability of this model was assessed in previous works, in which a masonry tower recently collapsed in Pavia (Italy) was analyzed and the predicted time to failure turned out to be close to the real one. The values of the parameters that define the model were obtained from experimental tests carried out on specimens taken from a part of the Cathedral contemporary to the tower. The numerical analyses point out the dangerousness of the tower conditions: indeed, a time to failure of about two centuries starting from today is predicted.

1. INTRODUCTION

Ancient massive masonry buildings, such as towers, can be subjected to stress concentrations which, if compared to the static (or short term) strength of the "material," may turn out to be not too high, but which can induce creep strains that can locally cause damage concentrations and severe damage effects. These concentrations are likely to have been the cause for the failure of the Civic Tower in Pavia (Italy), that occurred in 1989 [3], and, more recently, of the bell tower of the church of St. Magdalena in Goch (Germany).

After the occurrence of these failures, an extensive research program was carried out in Italy to assess the safety of many ancient masonry structures. This program included experimental tests, both destructive and non-destructive, in-situ and in the laboratory, structural monitoring, and numerical modeling [2]. On the side of modelization, much work has still to be done. Indeed, the complex mechanical behaviour of masonry is still far from being completely understood and, hence, simulated, even in the simpler case of monotonic load conditions.

Recently [6,7], a first attempt toward the modeling of the creep behaviour of rubble masonry and the associated damage effects was made by the authors, which proposed a mathematical model that accounts for creep strains, both reversible and irreversible, and damage effects; this model is briefly reviewed in Sec. 2. The implementation of the model in a commercial finite element code made it possible to numerically analyze the Civic Tower of Pavia and, upon calibration of the model parameters according to the available experimental data on the material, the time to creep failure of the tower could be reasonably predicted.

Here, the same model is employed to assess the safety of the belltower of the Cathedral of Monza (Italy), which exhibits severe damage phenomena. The main characteristics of this building will be presented in Sec. 3 and the results of the analyses will be illustrated in Sec. 4. In particular, the proposed numerical model allows one to capture the damage concentrations corresponding to the observed macrocracks in the tower. Also, the estimated time to failure is of the order of 650 years starting from the date of erection (around 1600), which suggests the necessity of performing extensive repair interventions to make the tower safer. Some final critical remarks conclude the paper.

2. MATHEMATICAL MODEL FOR THE CREEP OF MASONRY

Details on the proposed mathematical model can be found in [6,7]. Here, it is just worth recalling that, similarly to a previous viscoelastic theoretical model [1], the present one is based on the Burger's rheological model (Fig. 1), modified so as to incorporate damage effects.

The spring and dashpot in parallel form a Kelvin element, which is aimed to describe primary creep. In this element, only reversible viscoelastic strains develop and damage effects are not supposed to take place: thus, the stiffness of the spring, E^K, and the relaxation time of the dashpot, τ^M, do not change in time.

The spring and dashpot in series form a Maxwell element. The spring accounts for the instantaneous response of the material to a stress increment: a "static" damage variable, D^S, is introduced that affects the stiffness E^M of this spring and accounts for damage induced by an increase in stress. In the dashpot of the Maxwell element, strains evolve only when the stress exceeds a threshold, σ_0, corresponding in the model to the slip strength of the frictional (or Bingham) element. When σ_0 is exceeded, the secondary (or steady state) creep phase starts. Since "viscous" damage can develop in this element, according to the evolution of the damage variable D^V, the relaxation time τ^M of the dashpot is supposed to reduce progressively, so that the tertiary (or unstable) creep phase can start, according to the stress level, and creep failure can be described.

The evolution laws for the damage variables were obtained by modifying laws proposed by other authors for different rocklike materials (concrete, rocksalt).

Different laws are used in tension and compression, to match the so-called "uni-lateral" behaviour of most brittle materials, such as masonry.

The uniaxial version of the numerical model, proposed according to the rheological model, was extended by the authors to the general 3D case and implemented in a commercial finite element code (see Sec. 4); second-order damage tensors were introduced to account for damage-induced anisotropy. As already mentioned, the reliability of the model was assessed through the analysis of a collapsed masonry building (the civic tower in Pavia).

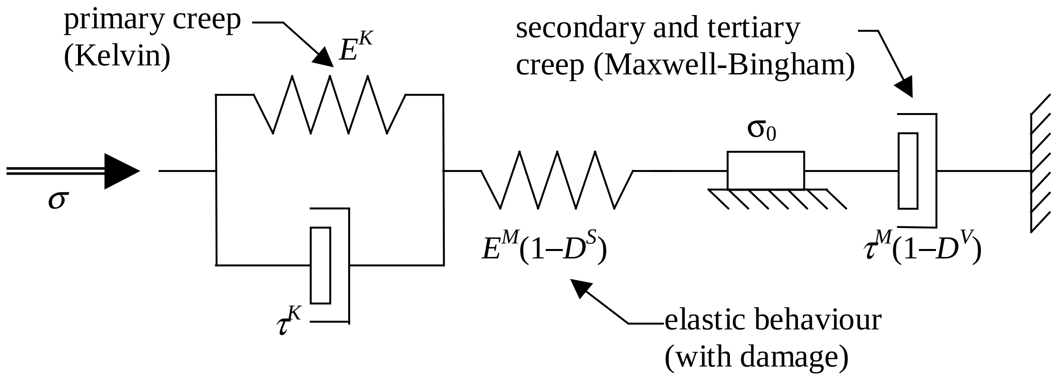


Figure 1 – Modified rheological Burger's model

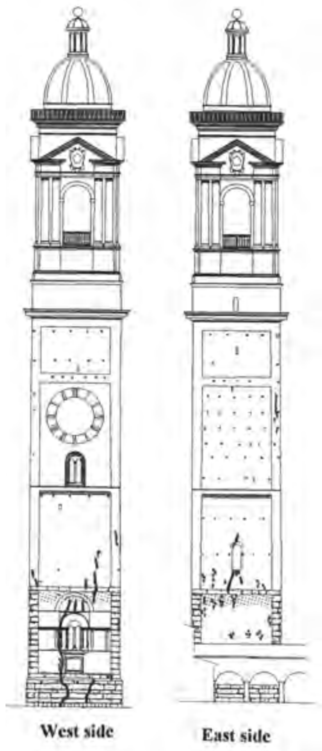
3. THE BELLTOWER OF THE CATHEDRAL OF MONZA

3.1 History of the tower. Crack monitoring

The Tower construction started in 1592, probably following a design of Pellegrino Tibaldi, and ended in 1605. The only serious documented damage that occurred to the Tower is a fire, developed in 1740 in the Bell-Tower, that caused the collapse of the bells and of the supporting frame down to the vault of the first floor at 11 m. In addition to other minor events, it is worth mentioning a thunderstorm in 1928, which caused the collapse of a pinnacle of the adjacent Cathedral, but no damages were reported. Nevertheless in 1927 some attempt was done with rough devices applied across the main cracks to detect their movements. The main crack in fact was already existing 1920.

No repair works have been done to avoid the propagation of the cracks, the main of them developing vertically and passing through the wall on the West and East facades of the Tower (Fig. 2b).

Since 1978 the cracks have been surveyed with removable extensometers showing a slowly increasing of their opening in time. Fig. 3 shows the monitored opening of the main crack from 1978 to 1997, with a clear tendency toward a faster increase from 1988: this has raised a great concern regarding the safety of the Tower itself. Other cracks can be seen from the internal walls of the Tower; they are vertical, very thin, diffused along the four sides of the Tower, and deeper near the entrance where stress concentration is higher. The cracks mainly occur across the bricks and were found to go 450 mm deep inside the masonry walls.



(a) (b)

Figure 2 – (a) Tower and Cathedral of Monza; (b) west and east façades

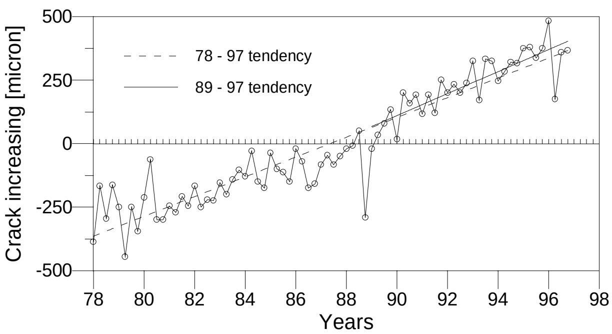


Figure 3 – Survey of the movements of the main passing through crack from 1978 to 1997

3.2 Geometrical survey

Owing to shortage of time and money, a photogrammetric survey of the external facades and a traditional survey in the interior were made. A geodetic net set up in the square before the Cathedral in 1993 was used as support; based on some points of the net, some significant points of the west facade were surveyed and used to straighten and return a series of photographic images [4]. No relevant leaning was measured due to the small subsidence, which is taking place in the square. As a result, a detailed three-dimensional model was obtained, from which the external and internal prospects and the sections for the internal prospects were obtained, in addition to a simplified model featured only by the geometrical aspects essential for structural analysis purposes.

As it was mentioned above, owing to the presence of main running cracks a complete crack pattern survey on the walls of the Tower. Cracks were surveyed visually and photographically, and reported on plan prospects and sections. In the meantime the measurement of the main cracks continued, and an automatic monitoring system was set up. The computed tendency to opening of the three surveyed cracks was 30.6, 31.3 and 39.7 μm/year, 1978 to 1995. Actually, if this tendency were considered from 1988 to 1997, the values change respectively to 41.2, 35.2 and 56.2. Special attention should be paid to the vertical cracks at the edges of the northern and southern facades of the Tower; they unfortunately cannot be monitored without scaffolding from the exterior of the building.

3.3 Material properties

Mortars and bricks sampled from the Tower were subjected to chemical, mineralogical-petrographical analyses and physical and mechanical tests.

Mortars are mainly based on putty lime and siliceous aggregates, coming from the near Ticino River. They are very weak, do not show good cohesion, so they could not be subjected to mechanical tests.

Two types of bricks are found in the Tower, different in colour (brown and light red) and in physical and mechanical properties. Red bricks, with the poorest properties, are the most diffused ones in the construction; this affects the weakness of the Tower. The strength of these bricks ranges between 4 and 12 N/mm^2 and the modulus of elasticity between 500 and 1330 N/mm^2.

3.4 Mechanical tests

Single and double flat jack tests were carried out to measure the value of the vertical compressive stress and the stiffness of the material [5]. Seven single flat-jack tests were performed, respectively, at heights of 5.4, 5.6, 13.0, 14.0, 31.5 and 38.0 m in the Tower, and the detected stress values are reported in Fig. 4. It is possible to see that, as expected from analytical calculations, the stress values increase from the top to the bottom of the Tower. The highest values seemed to be particularly dangerous, taking into account the strength values detected usually on this type of masonry.

Therefore some double flat-jack tests were carried out in order to check the mechanical behaviour of the masonry under compression. According to the stress levels at which masonry started cracking in the four cases, the state of stress has to be considered unsafe, if the safety factors adopted by the codes for new masonry were taken into account.

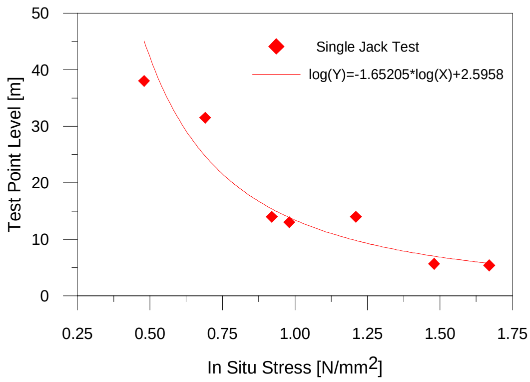


Figure 4 – Test point level against measured stress values

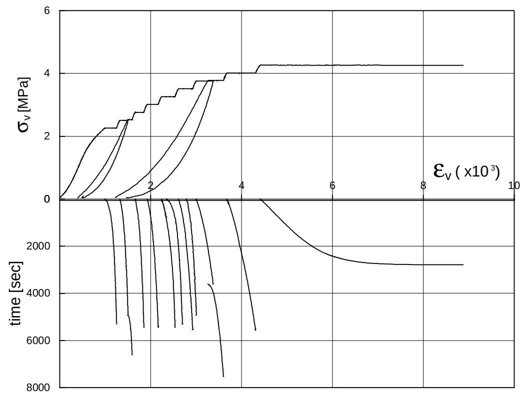


Figure 5 – Result of a test with subsequent load steps carried out on a prism from the crypt of the Cathedral

In addition to the in-situ flat jack tests, some laboratory tests were carried out on two large pieces of wall recovered during the opening of a door from the crypt of the Cathedral. The crypt was apparently built during the same period and reasonably likely with the same technique as the Tower.

Prisms of about 200×200×500 mm were cut and subjected to three series of uniaxial compression tests of different type after having been characterised through sonic tests. Initially, monotonic tests were carried out to have a first indication on the compressive strength of the masonry. Then cyclic tests were carried out, during which cycles of ± 0.15 N/mm^2 at 1 Hz were applied at increasing stress levels. Finally, compression tests were also carried out applying the load in subsequent steps and keeping it constant for a given time interval (about 1.5 hours). Because any single test lasted more than one day, the samples were unloaded before night, for safety reasons, and reloaded the day after.

In Fig. 5 plots are shown of the vertical stress vs. vertical strain and of the vertical strain vs. time, for a test with subsequent load increments. Creep strains can be clearly observed while the load is kept constant, with the appearance of tertiary creep during the application of the last load step.

4. FINITE ELEMENT ANALYSIS OF MONZA TOWER

Only the lower part of the tower was discretized by finite elements, for a height of 35 m. The finite element model employed to analyze the Monza tower consists of isoparametric eight-noded three-dimensional finite elements with three translational degrees of freedom each. The total number of d.o.f.s of the mesh is about 18000. A general-purpose commercial finite element code (ABAQUS) was used for the analyses. It is endowed with an interface that allows the user to develop subroutines to implement special constitutive laws.

In the numerical analyses, the only load acting upon the tower was supposed to be the self-weight. The unit weight of the material was taken equal to 18 kN/m^3. The upper part, not discretized by finite elements, enters the analysis only through its self-weight. The volume of this part and the belfry is about 1500 m^3, corresponding to a weight of 2700 kN; since the area of the cross section of the tower at 35 m is 53.54 m^2, a uniform vertical stress of 0.507 MPa was applied at the top of the f.e. model.

The values of the parameters defining the proposed damage model were identified according to the results of accelerated creep tests carried out on 3 specimens taken from the crypt of the tower.

In Fig. 6 the results of the numerical analyses are presented in the form of contour plots of one of the viscous damage variables that characterize the model, at different times. t=0 corresponds to the end of the building phase of the tower: viscous damage is supposed to start at this time with values equal to the static damage induced by the monotonic increase in stress during the building phase. The distribution of static damage does not significantly differ from that of viscous damage at t=150 years (Fig. 6a); in this time interval, damage is actually negligible. Fig. 6c refers to the estimated damage distribution at the end of this century (around 2100 A.D.). It is interesting to note that damage evolves in time mainly in the vicinity of the openings located in the West and East sides of the tower, thus matching the monitored cracks visible in Fig. 2b. The diffusion of damage is relatively slow up to 300 years, whereas it accelerates after this time (corresponding to the end of XIX century) leading the tower to failure after about 650 years. Fig. 6e shows the deformed finite element model at the predicted time to failure: note that the “barreling” of the lower part is matches by the spalling of the corners of the tower observed in the reality.

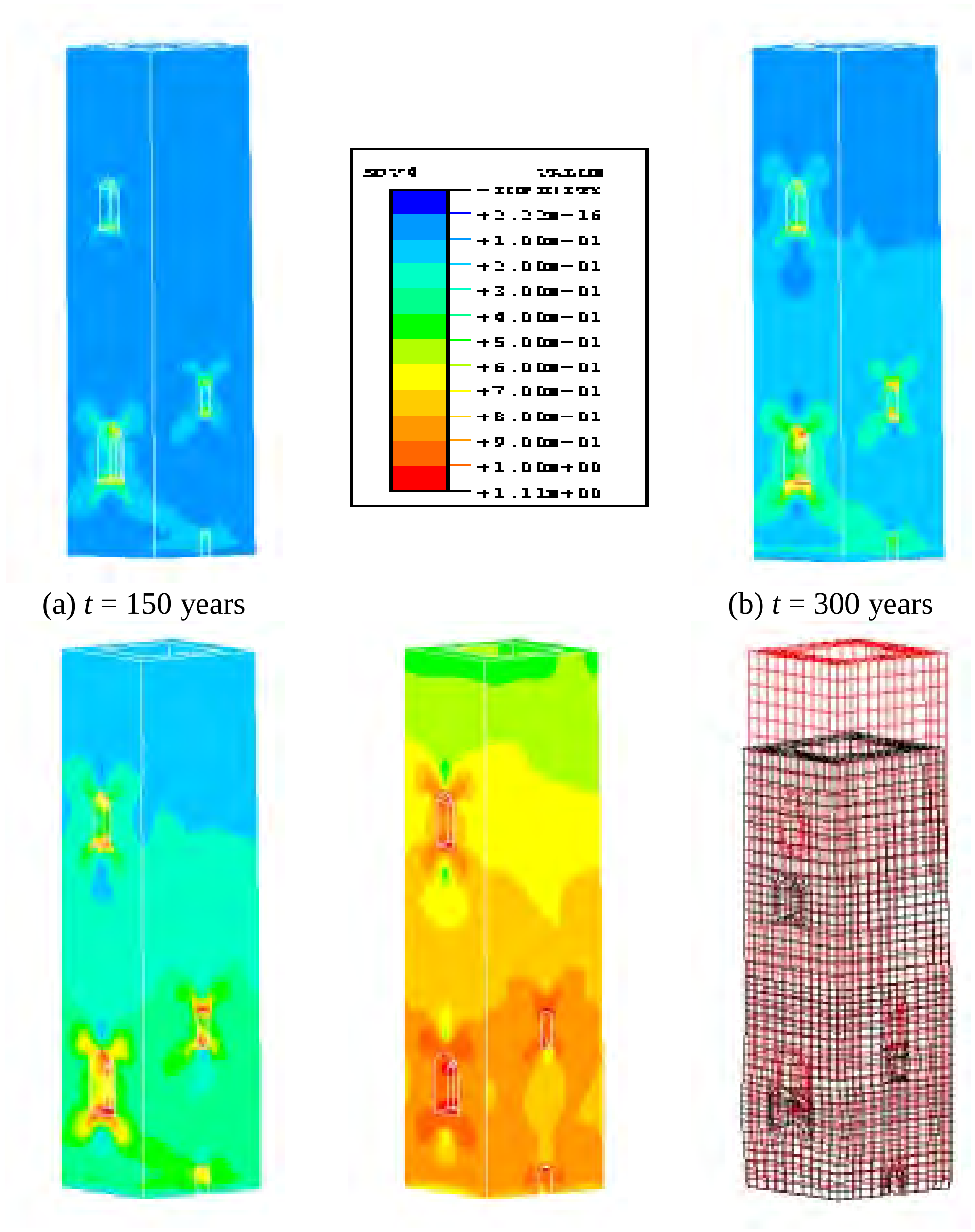
To emphasize the potentially catastrophic increase in damage with time, in Fig. 7 the horizontal displacement components of a couple of nodes located at the level of the most damaged zone are plotted versus time. Note how in these plots the three stages typical of the creep behaviour of materials subjected to damage are visible, namely primary, secondary (or steady-state) and tertiary creep; the latter phase precedes structural failure.

Note that the predicted time to failure of the tower is much shorter than the time to failure of a material element subjected to a uniaxial compressive stress of 1.23 MPa (Fig. 8): this can be explained according to the stress concentrations associated with the stress redistribution that follows the diffusion of damage throughout the tower.

5. CONCLUSIONS

As a conclusion of the experimental and numerical research carried out, some remarks are outlined in the following.

The damage caused by the dead load in heavy buildings as towers, defensive walls, church pillars, etc. cannot be neglected in the long term. It has now been proved that masonry in these cases can exhibit a creep behaviour, which can bring



(a) t = 150 years (b) t = 300 years

(c) t = 500 years (d) t = 650 years (e)

Figure 6 – Contour plots of the one of the principal components of the viscous damage tensor at (a) 150 years, (b) 300 years, (c) 500 years and (d) 650 years (estimated failure); (e) deformed f.e. mesh at failure

the building to partial or total collapse. Modeling this behaviour is still a quite difficult task, due to the complex mechanical behaviour of masonry, which is far from being completely understood. This difficulty is increased by the numerous masonry typologies, which prevents the possibility of conceiving one model for masonry; therefore several models need to be developed according to the nature of this composite material.

Furthermore, models have always to be calibrated on the basis of experimental results. This was done by the authors in analyzing the reasons for the collapse of the Civic Tower in Pavia. The implementation of this model allowed the authors to follow the first, secondary and tertiary creep phases, to reliably express the damage distribution and to match the time to failure of the structure.

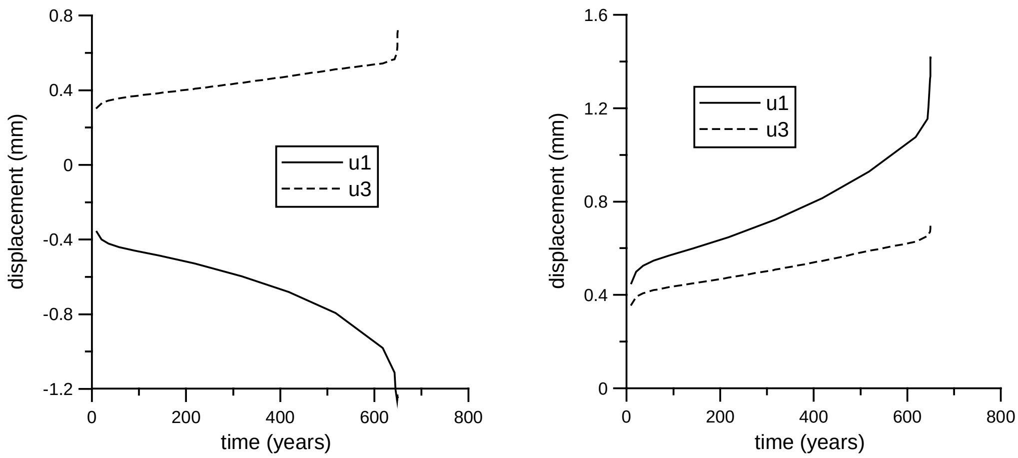


Figure 7 – Horizontal displacement components vs. time for two nodes of the finite element mesh located at a height of about 9 m

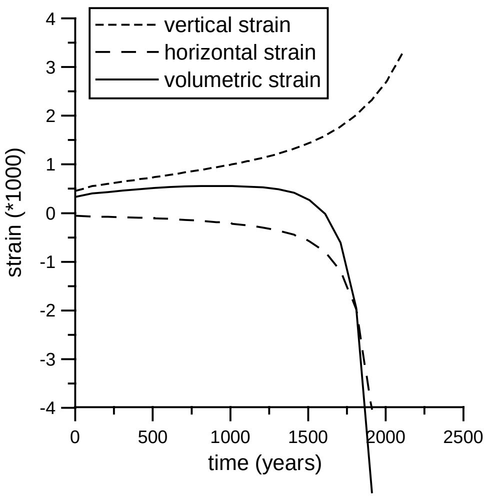


Figure 8 – Vertical, horizontal and volumetric strain vs. time for a material element subjected to a uniaxial compressive stress of 1.23 MPa.

Here, the application of the same model, upon calibration with data of experimental tests carried out on site and in laboratory, allowed the authors to match the actual location of the damaged areas and to make a reasonable prediction for the time to failure of the structure.

The obtained results stimulate further progresses in the future along this line of research, including improvements and a more effective implementation of the proposed mathematical model which, in conjunction with on site survey and investigation, can be used for the prevention of damages and collapse.

ACKNOWLEDGEMENTS

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NUMERICAL ANALYSIS AS A TOOL TO UNDERSTAND HISTORICAL STRUCTURES. THE EXAMPLE OF THE CHURCH OF OUTEIRO

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ABSTRACT

Historical structures are particularly difficult to analyze due to the lack of mechanical data. Nevertheless, significant information can be obtained from numerical analysis. In this paper an example of the use of numerical analysis to understand the damage and design the strengthening of one historical structure is presented. It is shown that the clear understanding of the structural behavior and the reliable strengthening based on sophisticated tools of analysis reduces the extent of the intervention.

1. INTRODUCTION

The analysis of ancient constructions poses important challenges because of the complexity of their geometry, the variability of the properties of traditional materials, the different building techniques, the absence of knowledge on the existing damage from the actions which affect the constructions throughout their life and the lack of codes. In addition, restrictions in the inspection and the removal of specimens in buildings of historical value, as well as the high costs involved in the inspections and diagnoses, often result in reduced information about the internal constructive system or the properties of the existing materials.

Nevertheless, significant advances occurred in the last decade concerning the development of adequate tools for the numerical analyses of masonry structures. In particular, recent advances adopt micro-modeling strategies, e.g. [1], in which mortar and stone are modeled separately, and macro-modeling strategies, e.g. [2,3], in which masonry is modeled as a homogeneous anisotropic material.

These aspects call for qualified analysts that combine advanced knowledge in the area and engineering reasoning, as well as a careful, humble and time consuming approach. This paper provides one example that demonstrates the careful use of numerical analyses to tackle practical engineering problems.

2. DESCRIPTION OF THE CONSTRUCTION AND EXISTING DAMAGE

The church of Saint Christ in Outeiro (Bragança), in the North of Portugal, was built in 1698-1738. The structure is mostly made of local shale stone and thick lime mortar (rubble masonry), even if regular masonry (granite ashlars and dry / thin joints) was used in doors / windows frames and the façade, see Figure 1. For a complete report on this structure please refer to [4].

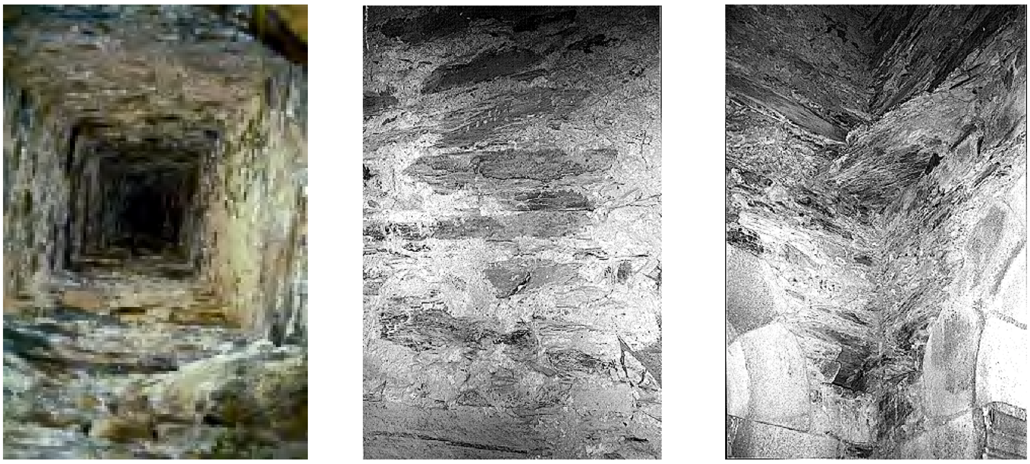


Figure 1 – Aspects of the irregular masonry in non-plastered areas, including ashlars around window openings

The structure is of moderate size (38×22 m^2 for the plan view, 13 m height for the nave and 22 m height for the towers), see Figure 2. The façade is a piece of large architectural value due to the false twin arch and the cladding in granite. It possesses a large opening and a balustrade that connects the bell towers. The side view shows framed galleries, forming three chapels. The interior of the church has a single nave (Latin cross) with crossed vaults, split in three parts by two arches. The arches are supported by the side walls and the transversal walls that divide the chapels, acting as buttresses.

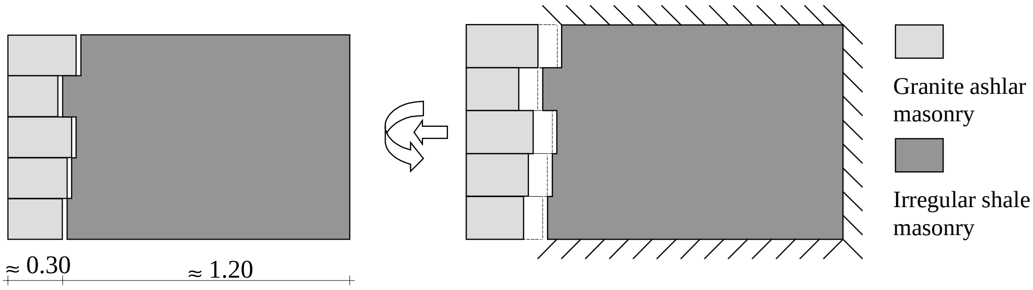
(a) (b) (c)

Figure 2 – Geometry of the church: (a) front view, (b) longitudinal section and (c) plan section

The damage of the structure is localized in the main façade and in the choir. Some additional minor cracking visible in other areas of the structure does not seem alarming. The façade features large movements, vertically, in the vicinity of the twin arch, and horizontally, towards the outside, around the opening and at the top central part, see Figure 3. The interior face of the wall does not shown any sign of horizontal displacements and, with the use of a boroscope it was possible to check that the wall is made of two leaves, separated by approximately 0.10 m in the vicinity of the opening, see Figure 4. Movements in the façade do not seem to have occurred recently.

(a) (b)

Figure 3 – Damage in the façade: (a) twin arch and (b) opening



(a) (b)

Figure 4 – Wall of the main façade: (a) geometry and (b) observed movements

The choir features heavy cracking of the arches and vaults, see Figure 5. It is separated from the façade, except in the brackets, and the columns are out of plumb 2.6% (around 0.08 m), see Figure 6.

3. STRUCTURAL ANALYSIS

In order to analyze the structure and justify the damage described, three different models have been adopted for the façade and one model has been used for the choir. Modeling, which resorts to the finite element method, included the linear elastic and the non-linear behavior of the material.

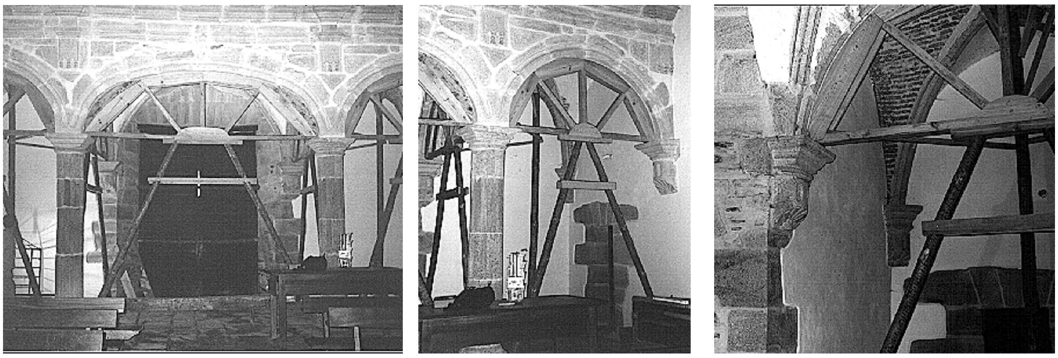


Figure 5 – Aspects of the choir's temporary support

(a) (b)

Figure 6 – Choir: (a) detail of separation above the main door and (b) columns out of plumb

3.1. Analysis of the Façade Subjected to Vertical Loading

For the analysis of the façade subjected to vertical loading a plane stress representation of the façade has been adopted. The thickness of the façade is given in Figure 7, in order to simulate the three-dimensional shape of the structure.

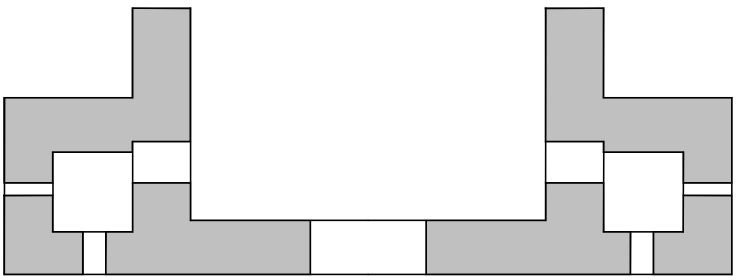
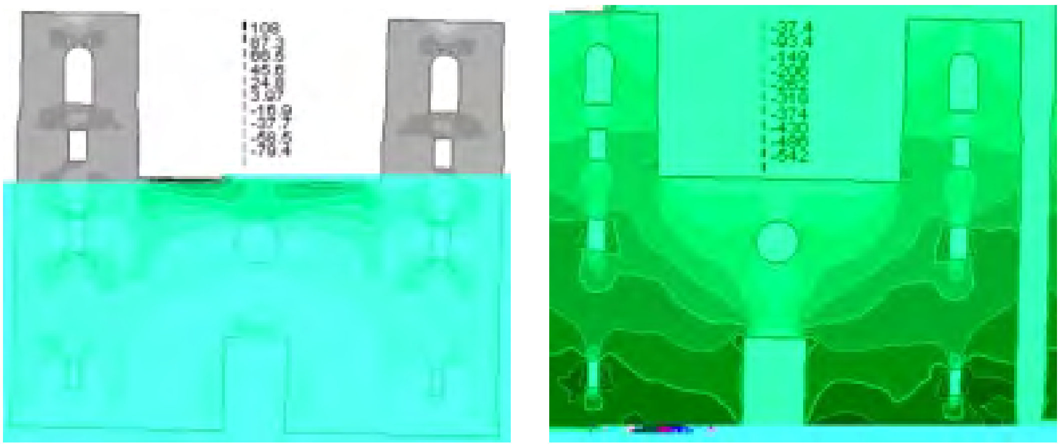


Figure 7 – Scheme of the adopted thicknesses for the plane stress model

The adopted elastic values were representative of the type of masonry found in the structure. The vertical loads considered in the analysis included the self-weight of the structure, the weight of the pyramidal roof of the bell-towers and the weight of the main nave roof. The soil-structure interaction has been modeled by interface elements, with properties obtained from in-situ testing of the soil.

The results of the analysis, assuming linear elastic behavior of the material, are given in Figure 8, in terms of maximum and minimum principal stresses. Under the most unfavorable hypothesis that all permanent loads are applied simultaneously and not in agreement with the construction phases, it is observed that the maximum value of the tensile stresses and the compressive stresses are extremely low, respectively +0.11 (tension) and -0.54 (compression) MPa.



(a) (b)

Figure 8 – Plane stress analysis for vertical loading – picture of the principal stresses (kPa) on the deformed structure: (a) maximum and (b) minimum

This analysis demonstrates that, under the assumption of a homogeneous material for the façade, the structure should not present any damage due to vertical loading. Indeed, the solid and robust structure of the church shows a clear over design. The localized damage of the structure, the signs that the movements of the structure have stabilized and the excellent conditions of the foundations / soil, indicate that the damage is due to seismic action. Earthquakes are normally unexpected in the North of Portugal but, in the vicinity of the structure, shakes were reported in 1751 and 1858 (Mercalli intensities VI and VII, respectively). For this reason, the analysis of the structure under combined actions (vertical and horizontal) seemed necessary.

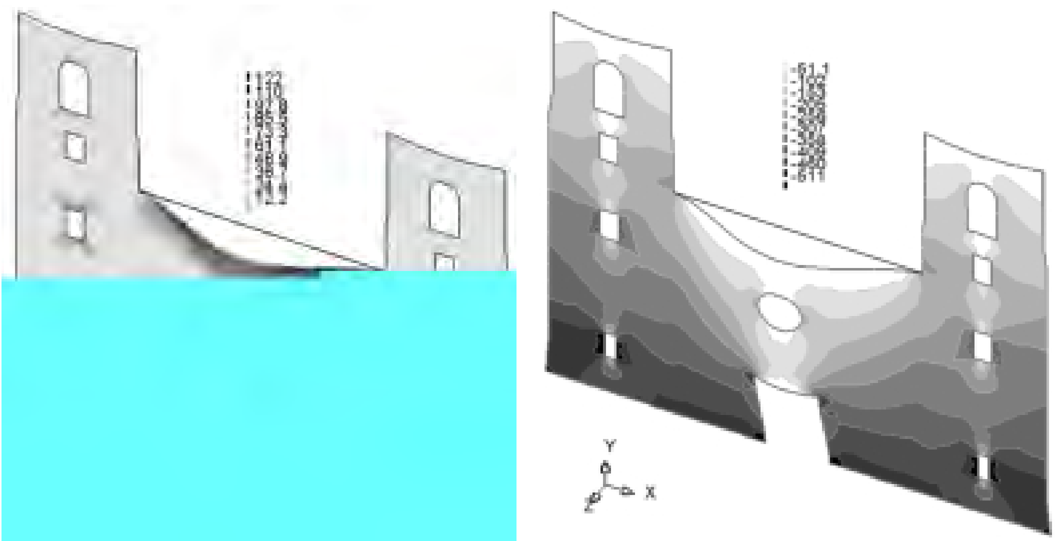
3.2. Analysis of the Façade Subjected to Vertical and Horizontal Loading

For this analysis, the model of the previous section was adopted but the plane stress elements have been replaced by shell elements. For the purpose of horizontal loading, it was assumed that the walls normal to the façade would act as shear walls, preventing any horizontal movement in this direction.

The vertical loading is the same as in the previous section. To assess the behavior of the structure under earthquakes, a simplified approach was used by replacing the seismic action by a set of equivalent horizontal loading. According to the Portuguese Code, the horizontal loading is given by 6.6% of the mass, for this particular region in Portugal.

Assuming that the top of the façade is capable of moving freely and adopting linear elastic behavior for the material, the results of the analysis are given in Figure 9, in terms of maximum and minimum principal stresses. It can be observed that the maximum value of the tensile stresses and the compressive stresses are still extremely low, respectively +0.12 (tension) and –0.51 (compression) MPa. Moreover, the maximum displacement normal to the façade is only 0.4 mm, which cannot be compared with the value of 0.10 m observed.

From these results it was concluded that the assumption of an homogeneous material is incapable of justifying the damage exhibited by the structure, even with the action of the horizontal forces equivalent to the seismic load. It is, however, stressed that the shape of the deformation shows some agreement with the observed deformation of the real structure.



(a) (b)

Figure 9 – Shell analysis for vertical and horizontal loading – picture of the principal stresses on the upper layer (kPa), on the deformed structure: (a) maximum and (b) minimum

The previous analyses demonstrated the need to represent the multiple leaves of the façade wall. For the purpose of a non-linear dynamic analysis, the wall was modeled with two different leaves, see Figure 4: granite ashlars with an average thickness of 0.30 m and shale masonry with and average thickness of 1.20 m, with their respective elastic properties.

Raleigh damping was assumed [5], in which the damping matrix $\underline{\underline{C}}$ is considered a linear combination of the mass $\underline{\underline{M}}$ and the stiffness $\underline{\underline{K}}$ matrices:

$$\underline{\underline{C}} = \alpha \underline{\underline{M}} + \beta \underline{\underline{K}}, \tag{1}$$

where the α and β coefficients are given by

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2 \frac{\omega_m \omega_n}{\omega_n^2 - \omega_m^2} \begin{bmatrix} \omega_n & -\omega_m \\ -1/\omega_n & 1/\omega_m \end{bmatrix} \begin{Bmatrix} \xi_m \\ \xi_n \end{Bmatrix}. \tag{2}$$

In this expression, ω_m is the first natural frequency and ω_n should be a higher frequency that significantly contributes for the dynamic response of the structure (the second lower frequency was adopted in this case). The usual damping coefficient ξ equal to 5% was adopted and a base acceleration was generated according to the Portuguese Code, see Figure 10a.

As the dynamic properties of the two leaves differ substantially, separation of the leaves will occur. The response of the structure, in terms of separation (i.e. relative displacement in the *z* direction) of the top edge of the façade is illustrated in Figure 10b, where it is confirmed that separation of the two leaves reaches very large values (around 0.09 m), which is comparable to the observed separation.

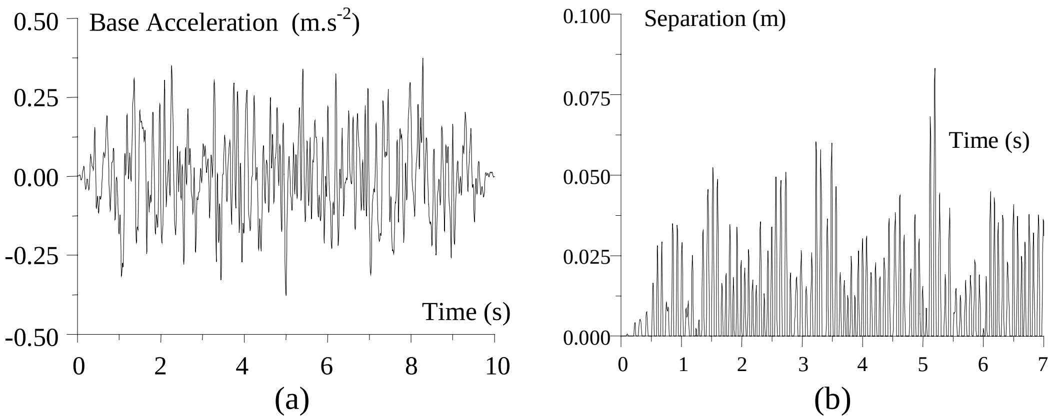
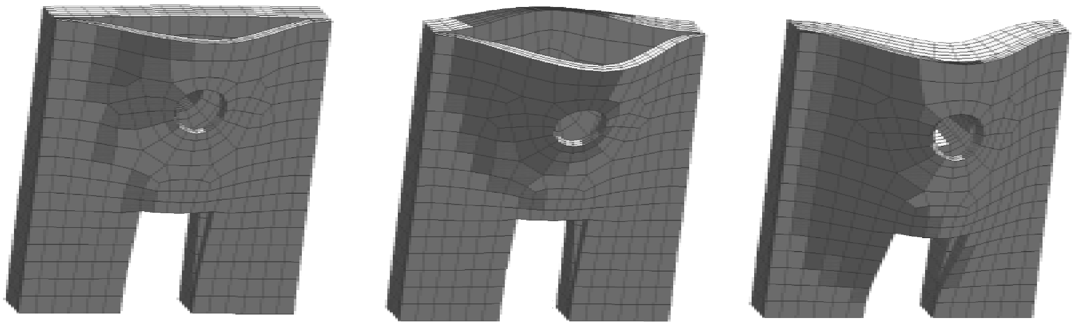


Figure 10 – Dynamic analysis with time integration: (a) artificially generated base acceleration and (b) separation between the two leaves

Figure 11 presents some examples of the deformed structure throughout time. It was possible to obtain different configurations of separation between the two leaves (in the top and center of the wall). For example, Figure 11a shows the shale wall practically in the original configuration and the granite wall moving out of plane. Figure 11b shows the walls moving in opposite directions. Figure 11c shows the walls moving in the same direction, with small separation at the top and large separation around the opening. This last form of separation seems to be in good agreement with the damage exhibited by the façade. Therefore, it seems possible to conclude that an earthquake was the main origin of this damage.



(a) (b) (c)

Figure 11 – Examples of deformed meshes through time

3.3. Analysis of the Choir

For the analysis of the choir, a simplified model was chosen in which vaults were represented by shell elements, arches and ribs were represented by beam elements and columns were represented by truss elements. The arches and ribs were supported in stone brackets.

Loading included all permanent loads and static horizontal forces equivalent to the seismic loading. Figure 12 illustrates the deformation of the choir under these loads, assuming linear elastic material behavior. The deficient original conception of the choir is clear, as it moves from the façade and side walls, even for vertical loading. If the vaults of the choir were originally connected to the side walls, or side arches were originally placed around the borders, the behavior would be monolithic. The values obtained for the maximum horizontal displacement is 0.006 m for vertical loading and 0.016 m for the seismic loading, amounting to a total of 0.022 m and confirming again the significant contribution of horizontal

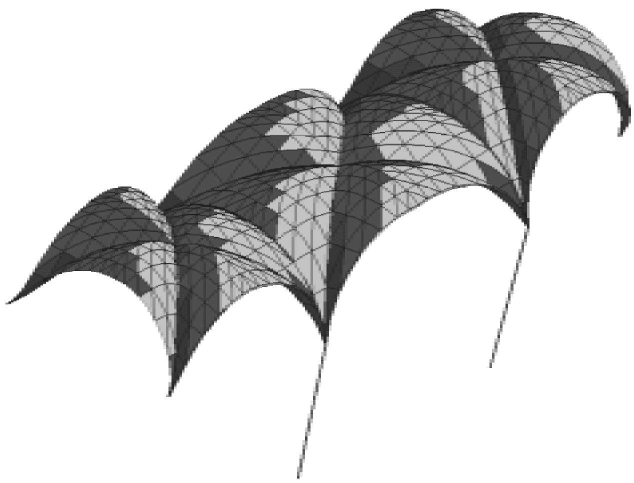


Figure 12 – Deformed mesh of the choir under vertical and horizontal loading

loading to the observed movements and damage. The value of the total displacement is well under the 0.08 m observed in the top of the columns but is in reasonable agreement with the separation between vaults and façade wall (around 0.015 m). The difference of displacement in the columns can be explained by the severe cracking of arches, ribs and vaults in the structure.

This simplified analysis demonstrates that the vertical load thrust is deficiently absorbed by the columns and a connection between the side walls and the vault is missing, meaning that the original conception of the choir is incorrect.

4. DESIGN OF THE INTERVENTION

The conclusions of the numerical analyses of interest for the definition of the intervention are that (a) the damage was due to seismic action and (b) there were two misconceptions in the construction of the church (façade and choir). Therefore, it is only necessary to correct these errors with proper façade wall tying and adequate connection between the choir and the external walls.

Keeping this conclusions in mind, the intervention was designed according to the modern principles of architectural heritage protection, namely, minimal repair, unobtrusiveness, removability (as reversibility is obviously intangible) and respect for the original conception, together with the obvious requirements of stability, durability and compatibility, see Figure 13. For the particular case, it suffices to disassemble and reassemble the façade external leaf with proper tying to the internal leave and to ensure proper monolithic behavior between the choir and the external walls. The latter was achieved with ties inside the vault filling.

It is noted that the selection of materials was careful, including the use of stainless steel, special mortars and non-intrusive grouting anchors (with sleeve).

5. CONCLUSIONS

This paper aims at demonstrating the possibilities and advantages of using advanced numerical analysis in the diagnosis and strengthening of the architectural heritage. For the particular case study, it was possible to conclude that: (a) the structure was already subjected to moderate earthquakes twice; (b) the structure is robust and conveniently tied, and the soil is of good quality, resulting in no need for a global intervention; (c) the main façade is a two-leaves wall, not conveniently tied and featuring partial separation, due to seismic action; (d) the choir structural misconception lead to heavy cracking and separation from the external walls, under seismic action. With the present numerical analysis it was possible to define an adequate and minimum intervention in the structure.

6. ACKNOWLEDGEMENTS

The authors acknowledge the partial support by project PRAXIS-C-ECM-13247-1998 funded by the Portuguese Science and Technology Foundation (FCT).

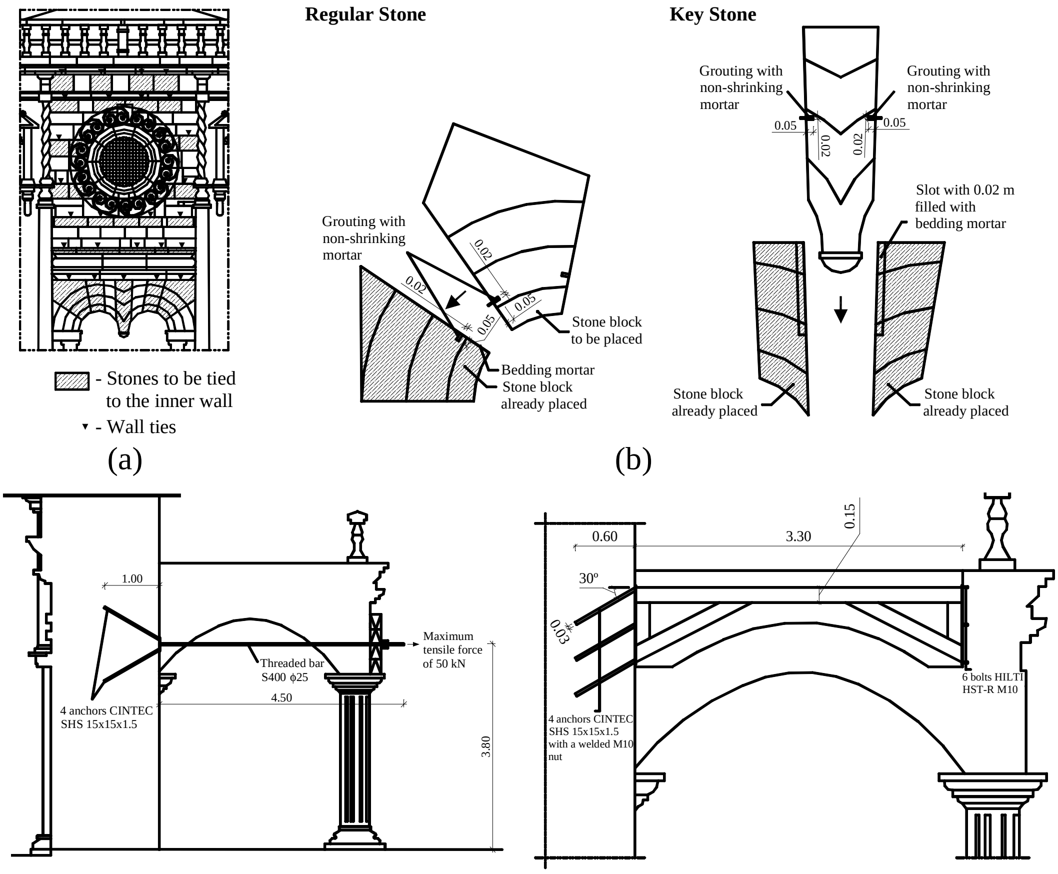


Figure 13 – Aspects of the designed intervention: (a) dismounting of the center of the façade and remounting with proper tying, (b) detailing of the twin arch dowels, (c) jacking operation to set the choir in its original position and (d) internal tying of the choir to the façade (additional tying of the vaults to the side walls was also carried out)

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RESEARCHES ON THE STABILITY OF ANCIENT CITY WALL IN XI'AN

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ABSTRACT

The stability of ancient city wall of Xi'an is researched in this paper. The twin shear stress strength theory is generalized to a twin shear elasto-plastic constitutive model in this paper. The twin shear elasto-plastic constitutive model is developed to simulate the behavior of materials where the uniaxial compressive strength is not equal to the uniaxial tensile strength. This model has been implemented into a special finite element program to analyse the strength and deformation of ancient Xi'an city wall, which was built in Ming dynasty about six hundred years ago, some was built in Tang dynasty about one thousand years ago. The parameters of materials were determined by triaxial compressive test with these rammed soils taken out of the city wall. The results are approximate to the case of the Xi'an city wall. Twin shear elasto-plastic constitutive model and the finite element program can be also generalized and applied to analyse the elasto-plastic deformation and strength of other engineering problems.

1. INTRODUCTION

The Mohr-Coulomb strength theory and Drucker-Prager criterion are the two best known strength theories (failure criteria) for the pressure-sensitive materials. The Mohr-Coulomb criterion is expressed in terms of maximum and minimum principal stresses and hence does not incorporate the effects of intermediate principal stress. The shortcomings of the Drucker-Prager criterion is that the trace of the failure surface on deviatoric planes is a circle which contradicts experiments of most pressure-sensitive materials, the approximation given by either the inner or outer cone or intersectional cone with Mohr-Coulomb failure surface to the true failure surface can be poor for certain stress combinations[1]. There are needs for better and improved failure criterion and elasto-plastic constitutive models for pressure sensitive materials. Some models were proposed in last two decades.

A new failure criterion for pressure sensitive materials, the twin shear strength theory, was proposed by Yu Mao-hong[2].

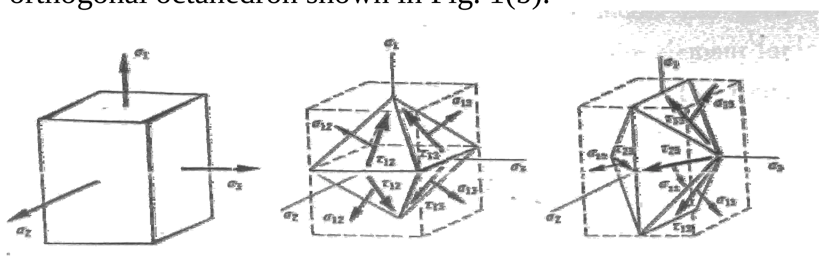
The twin shear stress strength theory assumes that the failure can be interpreted to begin when the sum of two larger principal shear stresses τ_{13}, τ_{12} (or τ_{23}) and the function of two corresponding normal stresses σ_{13}, σ_{12} (or σ_{23}) reaches a critical value. The expressions of twin shear strength theory can be formulated as follows:

$F = \tau_{13} + \tau_{12} + \beta(\sigma_{13} + \sigma_{12}) = C$, when $F \geq F'$ (1)

$F' = \tau_{13} + \tau_{23} + \beta(\sigma_{13} + \sigma_{23}) = C$, when $F \leq F'$ (1')

Where β is the coefficient which represents the effect of normal stresses on failure, and C is a strength parameter of material.

τ_{13}, τ_{12} and σ_{13}, σ_{12} are the principal shear stresses and the normal stresses acted on the sections of orthogonal octahedron are shown in Fig.1(a); τ_{13}, τ_{23} and σ_{13}, σ_{23} are also the shear stresses and normal stresses acted on the sections of orthogonal octahedron shown in Fig. 1(b).



(a) (b) (c)

Fig. 1. Stresses acted on the orthogonal octahedron

The magnitude of β and C can be evaluated by experimental results of uniaxial tensile strength σ_t and uniaxial compressive strength σ_c as follows

$$\beta = \frac{\sigma_c - \sigma_t}{\sigma_c + \sigma_t} = \frac{1-\alpha}{1+\alpha}, \quad C = \frac{2\sigma_c\sigma_t}{\sigma_c + \sigma_t} = \frac{2\sigma_t}{1+\alpha}, \quad \alpha = \frac{\sigma_t}{\sigma_c} \qquad (2)$$

The twin shear strength theory can be written in terms of the principal stresses σ_1, σ_2, σ_3 as follows.

$$F = \sigma_1 - \tfrac{\alpha}{2}(\sigma_2 + \sigma_3) = \sigma_t, \text{ when } \sigma_2 \leq \frac{\sigma_1 + \alpha\sigma_3}{1+\alpha} \qquad (3)$$

$$F = \tfrac{1}{2}(\sigma_1 + \sigma_2) - \alpha\sigma_3 = \sigma_t \text{, when } \sigma_2 \geq \frac{\sigma_1 + \alpha\sigma_3}{1+\alpha} \quad (3')$$

Twin shear stress strength theory is similar to the Mohr-Coulomb strength theory, but it takes the effect of the intermediate principal stress σ_2 into account, and agrees with some experimental results. The failure surface of the twin shear strength theory is a semi-infinite hexagonal cone with unequal sides[2], the size of failure surface is larger than the failure surface of Mohr-Coulomb theory.

2. TWIN SHEAR ELASTO-PLASTIC CONSTITUTIVE MODEL

Twin shear failure criterion can also be represented by stress invariant I_1 and deviatoric stress invariant J_2 and θ as

$$F = (1-\alpha)\tfrac{I_1}{3} + \tfrac{2}{\sqrt{3}}\sqrt{J_2}\cos\theta(1+\tfrac{\alpha}{2}) = \sigma_t \text{, when } F \geq F' \quad (4)$$

$$F = (1-\alpha)\tfrac{I_1}{3} + (\alpha + \tfrac{1}{2})\sqrt{J_2}(\sin\theta + \tfrac{1}{\sqrt{3}}\cos\theta) = \sigma_t \text{, when } F \leq F' \quad (4')$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2}\frac{J_3}{\sqrt{J_2{}^3}}, \quad 0° \leq \theta \leq 60°;$$

where θ is similar to the twin shear stress state parameter $\mu_\tau = \frac{\tau_{12}}{\tau_{13}}$ or $\mu'_\tau = \frac{\tau_{23}}{\tau_{13}}$ which was introduced by the first author of this paper[3].

The elasto-plastic matrix formulation for strain-hardening material and flow rule is similar to the Mohr-Coulomb's, but the yield function is changed from Mohr-Couomb criterion into twin shear failure criterion.

The flow vector is not uniquely defined at the corners of twin shear criterion located by $\theta = arcg(\frac{\sqrt{3}(1+\alpha)}{3(1+\alpha)}) + \frac{\pi}{6}$ when F=F', and the direction of plastic straining there is rounding off used by Owen and Hinton[1] for the Mohr-Coulomb criterion located by θ=0°and 60°.

3. STABILITY ANALYSIS OF ANCIENT XI'AN CITY WALL

The twin shear elasto-plastic constitutive model has been implemented in conjunction with a plane strain finite element for the deformation analysis of ancient Xi'an city wall, the finite element mesh is shown in Fig.2.

The parameter used for the rammed soils of Tang dynasty and Ming dynasty are determined by traxial compressive test.

Using the twin shear criterion and Mohr-Coulomb criterion, the plastic zones of city wall are shown in Fig. 3(a), (b) and Fig. 4(a), (b).

4. CONCLUSION

Twin shear strength theory in which the effect of intermediate principal stress is taken into account is generalized to a twin shear elasto-plastic constitutive model, and implemented into a special finite element program. This new model is applied to investigate the stability of the ancient Xi'an city wall. The results show that the plastic zones of twin shear elasto-plastic model is smaller than the plastic zones of MohrCoulomb's. The new twin shear elasto-plastic model may be applied to other problems.

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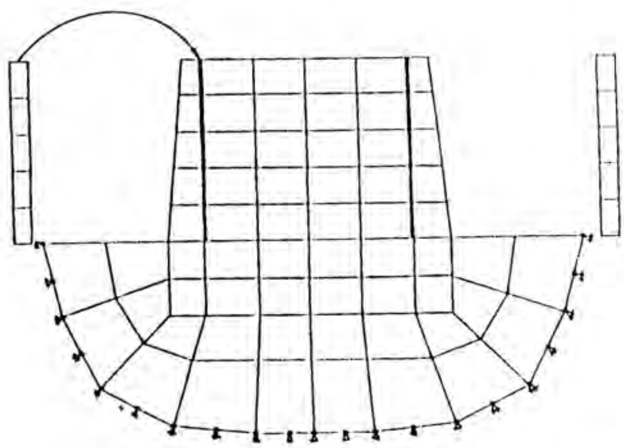

Fig. 2 Mesh of finite element

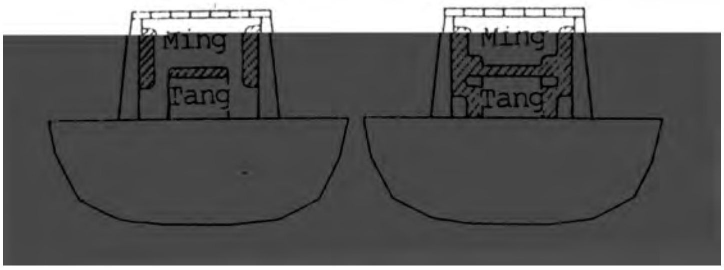

Fig. 3 Plastic zones of twin shear criterion (a) and Mohr-Coulomb criterion (b)

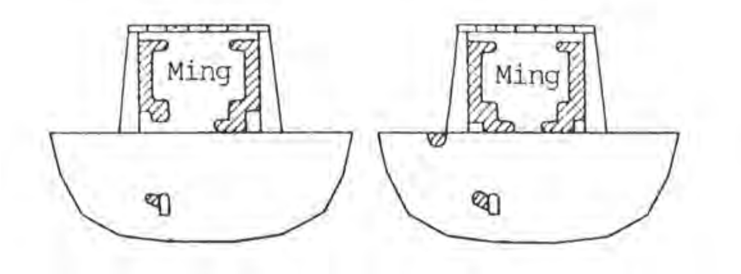

Fig. 4 Plastic zones of twin shear criterion (a) and Mohr-Coulomb criterion (b)

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DYNAMIC CHARACTERISTICS OF ANCIENT MASONRY CASTLE WALLS

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ABSTRACT

Generally the dynamic characteristics of stone wall structures depend on several factors such as contact, the type of interlocking bonding stones, and the filling materials. This paper describes a non-destructive technique for diagnosis of historic masonry stone structures using the measurement of natural frequency technique.

For this purpose, the castle wall of Nag-An Folk Town located in Sunchon, Korea was selected as a model. The Nag-An Town Castle is one of the well maintained historical remains constructed in the Chosun Kingdom of Korea. The construction started in 1397 A.D and was finished in 1626 A.D. The non-mortar castle wall is 1470m long and the average height is 4m with a width of 3 4m. The exterior of the wall is bonded with 1 2m rectangular rough-faced stone and the inside of the wall is filled with gravel. The traditional village still remains inside the Nag-An Town Castle, and they have a regional food festival every October.

Transverse vibrations were measured at 8 points around the castle. The measured natural frequency of the first mode was 26Hz 41Hz, and the shear modulus of filling material was 2.142 x 10^3 8.915 x 10^3 f/ . With these results, it may be assumed that the filling material is gravel or a sand-gravel mixture. It is expected that the information provided by this paper will be useful for addressing the maintenance problems of the old castle walls.

1. INTRODUCTiON

The old castle wall is one of the representative ruins among historic stone structures. The knowledge of structural (or mechanical) behavior of historic remains plays a key role both in the preservation and in the restoration of them. However, it is hard to predict the behavior of stone structures by the static analysis method, because of irregularity of face stones and also due to randomness of backfill materials. Inversely, one can estimate the mechanical characteristics and robustness of structural members or the system of stone masonry structures by measuring the dynamic characteristics. There were some studies conducted on the serviceability of arch bridges and the stiffness of partially fixed steel beams by measuring the natural frequencies [2,5,3]. The dynamic characteristics of stone castle walls mainly depends on the contact and interlocking types of face stones and backfill materials. When surveying the cultural heritages, preserving their original state is of primary importance, and so one should consider non-destructive methods first. This paper deals with a method of estimation of fundamental frequency and serviceability of ancient castle walls by means of a non-destructive technique. The estimated fundamental frequency of the stone castle wall can aid in the estimation of the shear modulus of the filling material as well as the overall stability of the wall. For this purpose, the castle wall of Nag-An Folk Town was selected as a model.

2. GENERAL INFORMATION ABOUT THE NAG-AN TOWN CASTLE WALL

2.1 General structure of the town castle wall

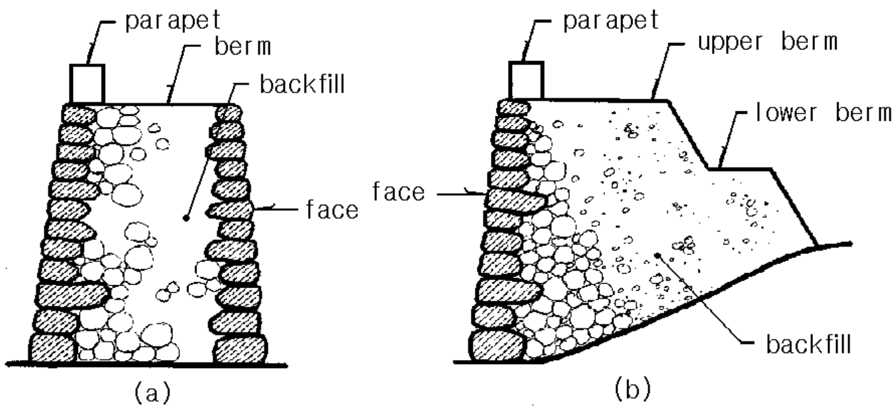


Figure 1. Section of stone masonry castle wall. (a) double faced wall, (b) single faced wall. The Nag-An Town Castle has a double faced wall.

There are two types of castle walls depending on whether the facing is single-sided or double-sided. Single facing is called ‘pyunchook’ and double facing is called ‘hyupchook’. The Nag-An Town Castle wall belongs to the ‘hyupchook’ category. Most castle walls in Korea consist of two major parts; one is a face of stone wall, and the other is backfill. The facing wall consists of rough faced rectangular granite, and backfill coarsed gravel, crushed rock or soil, etc. Stones used for the exterior face of the wall are somewhat elaborate and big, while stones used for the interior face are relatively less elaborate and smaller. Long depth stones and short depth stones were placed alternately, and facing stones and backfill material are well-interlocked. Large sized stones were placed in the lower layer and the relatively smaller stones were placed in the higher layer. So, the castle wall remains stable in an almost vertical state.

2.2 Nag-An Town Castle

Nag-An Castle which surrounds the Nag-An Folk Village of 0.224km^2 is located 22km west of Sunchon City, Cheollanamdo in Korea. The Nag-An Town Castle is one of the well maintained castles that was built in the Chosun Dynasty. It was constructed on the flat ground surrounded by mountains to guard the village from Japanese aggression. The construction started in 1397 A.D and was finished in 1626 A.D. Both the Nag-An Castle and Folk Village have been maintained as national cultural properties since the area was designated as Historic Site No.302 in 1983. The perimeter of the wall is 1470m long, the average height is 4m and the width is 3   4m. The face of the wall consists of non-bonded 1   2m rectangular rough-faced stones and the inside of the wall is filled with gravel. The wall of the eastern part still remains almost in its original form, but 50~60% of the rest of the wall was destroyed, so restoration of the castle wall was done in 1984. Inside of Nag-An Town Castle, the traditional folk village still remains. The folk village of 108 households is a living example of the folklore and history of ancestors. The Nag-An Folk Festival is held every May and the Namdo Cuisine Festival is held every October in this village.

Figure 2. Aerial view of Nag-An Castle and Folk Village.

3. THEORETICAL NATURAL FREQUENCY OF SYSTEM

3.1 Assumptions for the estimation of fundamental frequency

The natural frequency of a structure depends on its materials and geometrical properties, support and loading conditions. Monuments such as stone pagoda can be modeled on a multi-degree of freedom system. However, because of the small size of the aggregates for backfill compared to the dimensions of the whole structure, stone masonry structures such as castle walls can be modeled on a continuous system instead of a discrete system [4, 6].

Especially, Nag-An Town Castle can be thought of deep beam whose deformation is mainly governed by shear force. Deep beams are those in which the ratio of span length to the depth in the direction of vibration is not much greater than 1, or may even be less than 1. Shear deformations become an important part of deep beams. In other words, it is more dominant than flexural deformation [1].

3.2 Fundamental frequency of uniform shear beams

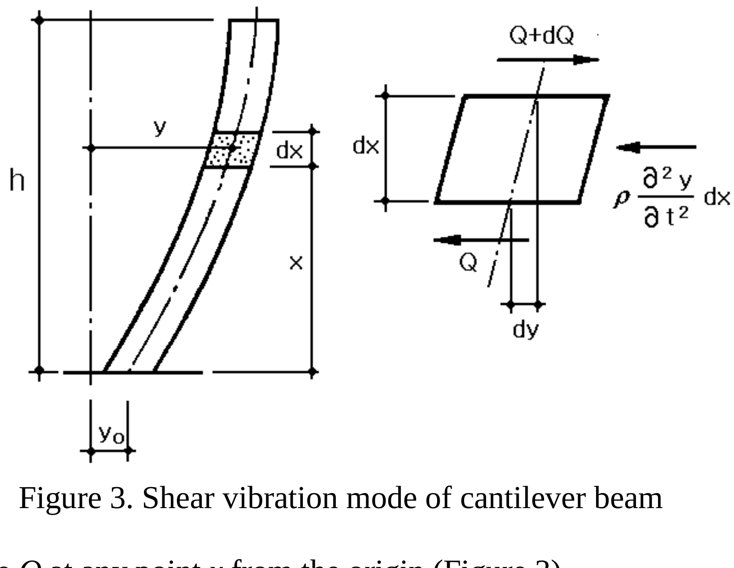


Figure 3. Shear vibration mode of cantilever beam

The shear force Q at any point x from the origin (Figure 3)

$$Q = GA\frac{\partial y}{\partial x} \tag{1}$$

Shear force at the point *dx* from point *x*

$$Q + \frac{\partial Q}{\partial x} dx = GA\frac{\partial y}{\partial x} + GA\frac{\partial^2 y}{\partial x^2} dx \tag{2}$$

According to D'Alembert's principle, the sum of these shear forces acting on element *dx* must be in equilibrium with the sum of the corresponding inertia force. So, the differential equation of motion takes the following form,

$$GA\frac{\partial^2 y}{\partial x^2} dx = \rho A dx \frac{\partial^2 y}{\partial t^2} \tag{3}$$

Where ρ is mass per unit volume. If one assumes the harmonic vibration of the system, the general solution of Eq. (3) is given by

$$y(x,t) = \{C_1 \cos(\omega x / c) + C_2 \sin(\omega x / c)\}\sin(\omega t), \quad c = \left(\frac{G}{\rho}\right)^{1/2} \tag{4}$$

The boundary conditions for integral constants C_1 and C_2 are,

$$\left[y\right]_{x=0} = 0, \quad \left[GA(\partial A / \partial x)\right]_{x=h} = 0 \tag{5}$$

From these boundary conditions, one can obtain the circular natural frequency ω_n, which denotes the angular velocity of the motion,

$$\omega_n = \frac{(2n-1)\pi}{2h}\sqrt{\frac{G}{\rho}}, \qquad (n = 1, 2, 3 \Lambda\ ) \tag{6}$$

The natural frequency expressed in hertz or cycles per second,

$$f_n = \frac{\omega_n}{2\pi} = \frac{(2n-1)}{4h}\sqrt{\frac{G}{\rho}}, \qquad (n = 1, 2, 3 \Lambda\ ) \tag{7}$$

Fundamental natural frequency corresponding to the first mode of vibration is obtained with n=1 in Eq. (7)

$$f_1 = \frac{1}{4h}\sqrt{\frac{G}{\rho}} \tag{8}$$

3.3 Relation between the backfill material and natural frequency

The velocity of the propagation of shear waves depends on the density and the shear modulus of elasticity of the ground layers through which they pass. The velocity of the shear wave v_S is,

$$V_S = \sqrt{\frac{G}{\rho}} \tag{9}$$

The velocity of the shear wave can be also obtained by using the natural frequency of Eq. (9). The velocity of the shear wave and shear modulus changes due to backfill material are shown in Table 1 [8].

Table 1. Velocity of shear wave and shear modulus of backfill material

Description	**Shear wave velocity (m/sec.)**	**Mass density (kgf/m^3)**	**Shear modulus (kgf/cm^2)**
Sand	60	1800	66
Reclaimed land	100	1800	184
Sandy clay	100~200	1800	184~ 735
Sand-bearing gravel	250	2000	1276
Moist sand	300~400	2000	1837~3265
Gravel	600	2200	8082

4. THE NATURAL FREQUENCY MEASUREMENT OF THE NAG-AN CASTLE WALL

4.1 Dimension of the castle wall at measurement point

When the variable stiffness of the wall is replaced by the constant mean stiffness, one can easily expect some errors. However, the error is less than 10 percent even for a stiffness ratio of maximum to minimum as high as 25 [7]. Hence, in most applications, a non-uniform masonry castle wall can be analyzed reasonably well as a uniform shear beam. The dimensions of wall are shown in Table 2 and Figure 4.

Table 2. Measured dimension of castle wall at measurement point

Point	**H1(mm)**	**H2(mm)**	**L1(mm)**	**L2(mm)**	**S1(mm)**	**S2(mm)**
1	3850	4210	3540	4240	400	300
2	3400	3080	3900	4200	100	200
3	3550	2900	4110	4300	0	190
4	3630	2370	3730	3950	0	220
5	3520	3900	5000	5700	200	500
6	3450	3950	5200	5200	0	0
7	3680	3720	4850	5600	350	400
8	3500	2700	3350	4030	280	400

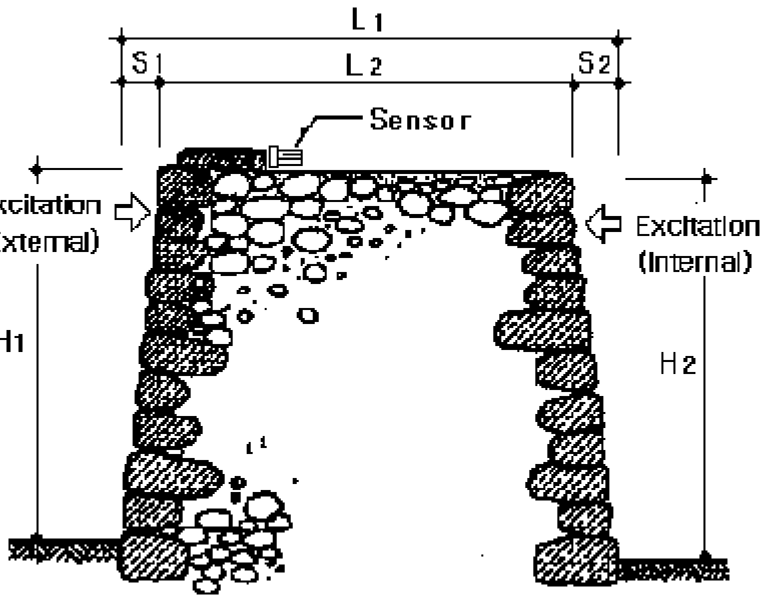


Figure 4. Sensor lacation and excitation point

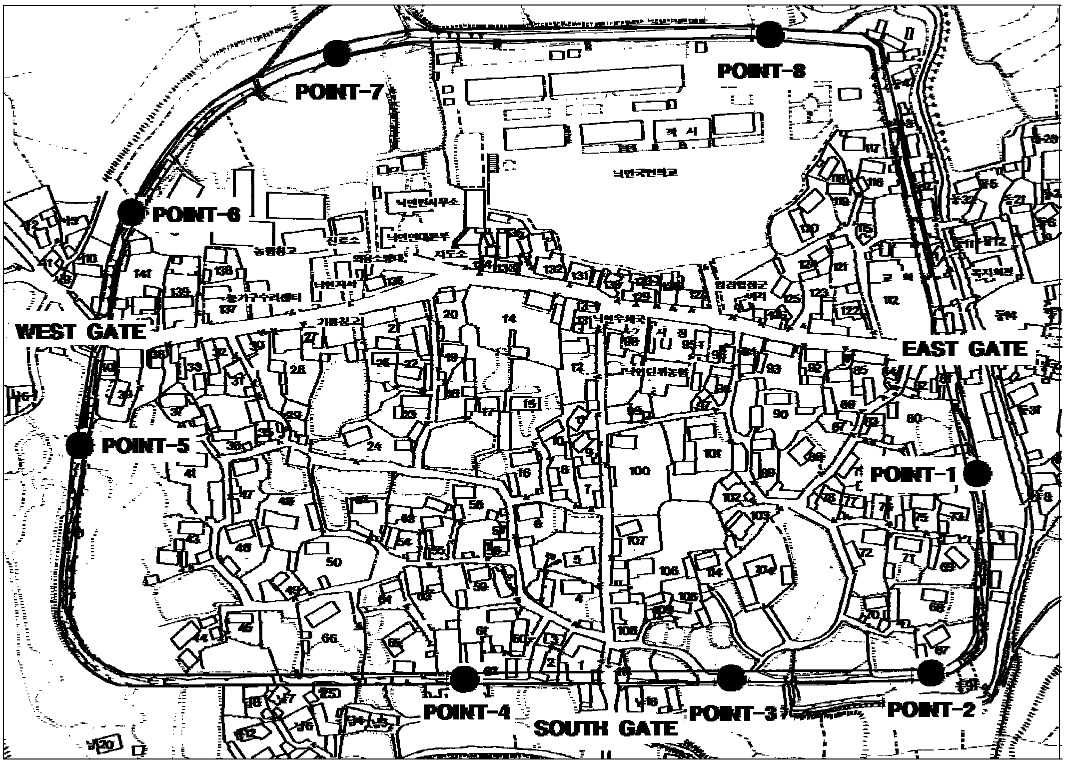


Figure 5. Layout of Nag-An Town Castle and location of measuring point

4.2 Measurement

Transverse vibrations were measured at 8 points around the castle. The locations of the measuring points are shown in Figure 4. Sensor locations and excitation points are also shown in Figure 4. If one uses a steel hammer for excitation, the energy density is not sufficient to excite the whole structure, and this may cause some damage to the cultural properties. For this reason, a rubber hammer is used in order to reduce the impact, which also helps to increase the pulse duration and to diminish the frequency range covered by the energy spectrum. The appliances used for measurement were B & K 8318 for the accelerometer, Sony 8 Channel for the D.A.T. and B & K 2133 for the frequency analyzer.

4.3 Test results

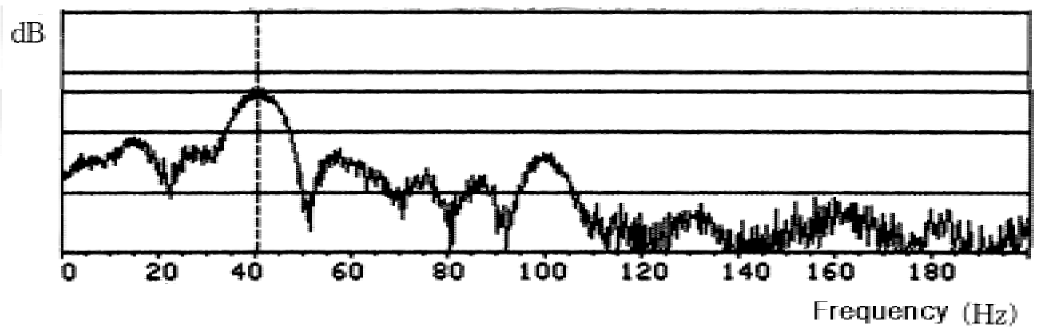
Figure 6 shows the examples of the frequency spectrum obtained by Fast Fourier Transform analysis, which aids in the estimation of the natural frequency of vibration. The test results which were obtained from excitation on the internal face of the wall were less influenced by the excitation force than those obtained from excitation on the external face of the wall. This was result of the sensor installed on the base of parapet that was near to the external face of the wall.

The measured fundamental natural frequencies and calculated shear modulus of the Nag-An Castle Wall are shown in Table 3. The natural frequencies were in the range of 26Hz 41Hz, and the shear modulus of backfill material was in the range of 2.142×10^3 8.915×10^3 f/ . With these results, one can predict that the backfill material is gravel or a sand-gravel mixture. Actually, an officer who was in charge of the restoration project of the Nag-An Town Castle, ascertained that the castle wall was restored using 50~100mm gravel for backfill.

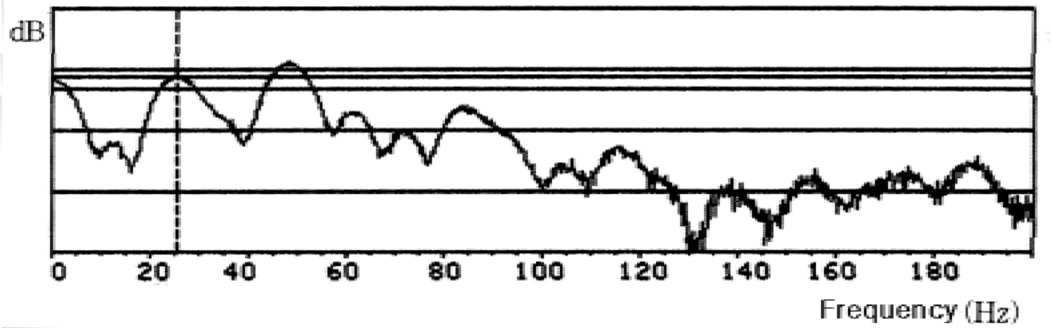
To apply the present results of the study to other historic properties, initial values must be prepared first. However, the coefficients of friction of stones critical to decision of stiffness constants depend on several factors, i.e. temperature, velocity of sliding and amount of slip of stones, roughness of contact face, cracks etc [9]. Therefore, more advanced study should be done and lots of measurements should be taken and analysis data should be prepared.

Table 3. Test results

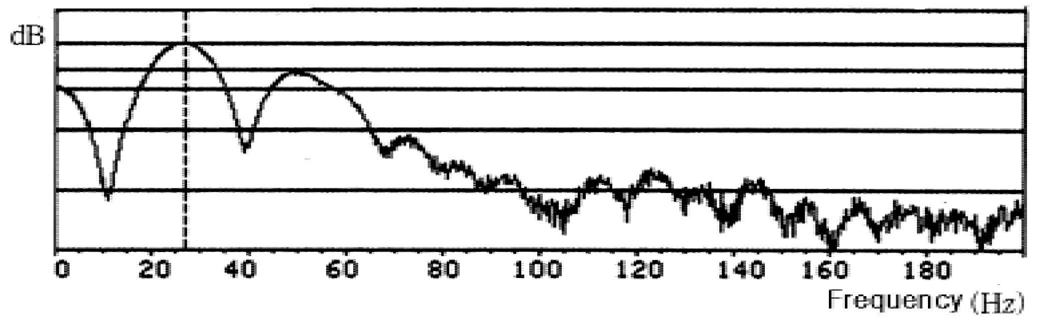
Point No.	**Wall Height(mm)**	**f_1 (Hz)**	**G (kgf/cm^2)**	**Exitation**
1	4,030	41	8.915×10^3	exterior wall
2	3,240	26	$2,317 \times 10^3$	exterior wall
3	3,225	26	$2,296 \times 10^3$	interior wall
4	3,000	27	$2,142 \times 10^3$	interior wall
5	3,710	27	$3,276 \times 10^3$	interior wall
6	3,700	35	$5,476 \times 10^3$	interior wall
7	3,700	29	$3,759 \times 10^3$	interior wall
8	3,100	27	$2,288 \times 10^3$	interior wall



(a) Measure point 1



(b) Measure point 3



(c) Measure point 5

Figure 6. Frequency spectrum obtained from Fast Fourier Transform analysis

5. CONCLUSION

It is difficult to analyze historic stone structures by using static methods because of the irregularity of wall stones and the randomness of backfill materials. By using the dynamic characteristics, however, one can estimate the behavior and stiffness of structural systems as well as evaluate the safety or robustness of the structures. With this point in mind, Nag-An Castle Wall was selected as a model. The results can be summarized as follows:

First, the proposed non-destructive method for the evaluation of historic stone structures can be applied to the estimation of shear modulus of backfill material and the overall stability of wall itself.

Second, the measured fundamental natural frequency of the Nag-An Castle Wall was in the range of 26Hz 41Hz, and the shear modulus of the filling material was in the range of 2.142 x 10^3 8.915 x 10^3 f/ . On the basis of these results, one can predict that the backfill material is gravel or a sand-gravel mixed type. Actually, the castle wall was restored using 50~100mm gravel for backfill material.

Third, to be able to apply the results of present study to other historic properties, initial values by non-destructive methods are a prerequisite. However, because the measured values depend on several external factors, more advanced study should be done.

Finally, it is expected that the information provided by this paper will provide assistance in the maintenance problems of all historic stone structures as well as old castle walls.

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STABILITY OF TILTED MASONRY WALLS UNDER SEISMIC TRANSVERSE FORCES

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ABSTRACT

There are many different geometrical imperfections in ancient masonry structures. Tilting of the walls and columns due to differential settlement of foundations, and reduced areas of contact at the base of the walls and columns due to edge damage are some of them. It is evident that such imperfections reduce the stability of masonry walls and columns under earthquake excitations. In this paper, particularly, the effect of tilting on the stability of masonry walls under seismic transverse forces is investigated. For this aim, a numerical model developed by La Mendola and Papia to investigate the stability of masonry piers under their own weight and an eccentric load is utilized and adapted to the problem at hand. The analysis is carried out for various angle of tilt. It was shown quantitively that tilting reduce the stability of masonry walls significantly.

1. INTRODUCTION

Besides wood, masonry is the most important construction material in the history of mankind. Masonry has been used, in a wide variety of forms, as a basic costruction material for public and residential buildings in the past several thousand years. Small daily life buildings, sheltering facilities, houses, shops etc. were not preserved carefully and mostly vanished. Whereas more social attention was paid to major sized public buildings (temples, theaters, agoras, large public baths etc.) because of their symbolic character. In recognition of their importance and value, many of those buildings have been ranked among the assets of highest category of mankind's historical and cultural heritage [1,6].

It is well-recognized now that many of the great civilizations since from the early years of history settled in earthquake prone zones. This is a consequence of the fact that most convenient geographical locations for building a city are valleys and cross-roads, which frequently follow the locations and intersections of active seismic faults. As proven by the historical data, many ancient and medieval towns and cities have already been destroyed by earthquakes. Some of them have been rebuilt at the same site, while others have been relocated to avoid future possible seismic impacts [1,6].

Ancient structures are subject to ageing effects and many different "imperfections" have formed in these structures in the course of time. Material deterioration caused by weather action, tilting of the walls and columns due to differential settlement of foundations, and loss of contact area at the base of the walls and columns due to edge damage are only some of them. For example, a large number of columns of the Temple of Apollo at Bassae (S. Greece) in their present condition are tilted due to differential settlements of the base slab, and many drums have broken corners [4,5]. It is obvious that such imperfections reduce the stability of masonry walls and columns under earthquake excitations. The effects of several imperfections could be additive and the cumulative effect of many imperfections may render, especially, deteriorating abandoned ancient structures vulnerable to earthquakes.

In the present study, the effect of tilting on the stability of masonry walls subjected to seismic transverse forces is investigated. To this end, a numerical solution procedure developed by La Mendola and Papia [2] to investigate the stability of masonry piers under their own weight and an eccentric load is adapted to the problem at hand.

2. ANALYSIS MODEL

For a masonry wall, especially when the wall is very slender, the most unfavorable condition can occur when the seismic-input direction is orthogonal to the plane of the wall itself, because the large deflections, consequent to the small lateral stiffness, amplify second-order effects, implying an instability risk.

The effects of any connections with other walls in the direction orthogonal to that of the wall considered can conservatively be neglected, especially when the wall itself has a large horizontal length. Therefore, in order to verify the safety condition, the strip of tilted wall located at the middle span can be considered as a fixed-free ended prismatic member.

Figure 1(a) shows a tilted masonry wall strip of length L and thickness H. The wall strip is ideally divided into n elements, all having the same length $l = L/n$, numbered from one to n, starting from the top end, and delimited by $n+1$ sections, numbered from zero to n [2]. In the figure, P indicates a concentrated vertical load transmitted to wall by the supported deck or dome, if any, and W is the total weight of the wall strip. The seismic coefficient c determines the values of the transverse forces on the basis of the expected earthquake intensity. Each element is affected by the weight W/n to which the inertia force cW/n corresponds. Both these forces are assumed to be concentrated and applied to the center of gravity of the element. Figure 1(b) shows the vertical equivalent of the tilted wall strip.

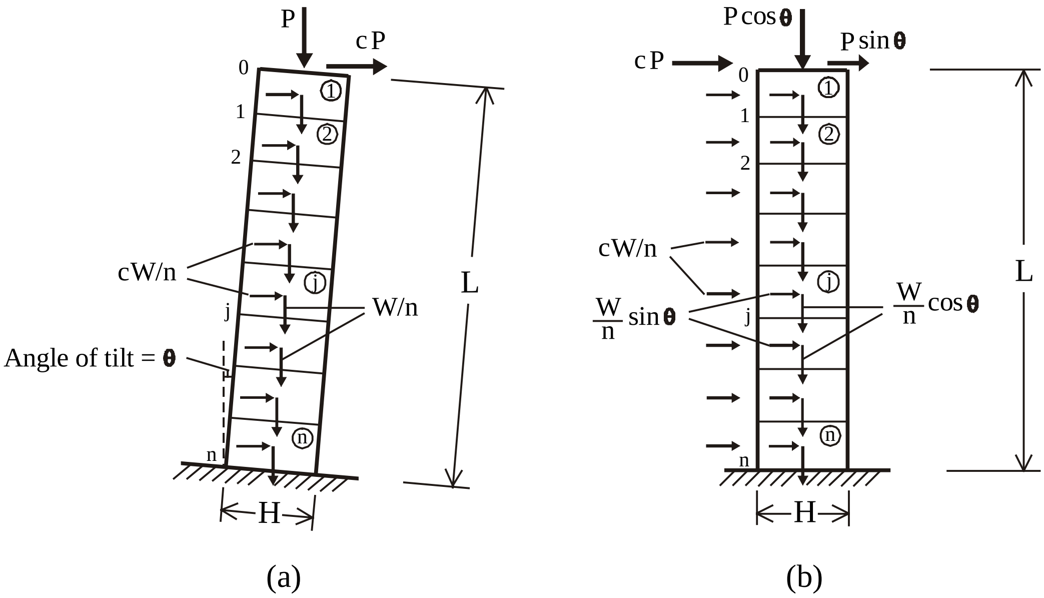

Figure 1. (a) Geometry and load condition of a tilted masonry wall strip, and (b) Its vertical equivalent

The deflected shape of the vertical equivalent wall strip under considered loading condition is shown in Figure 2. It is assumed that the curvature of each element is uniform, and is defined by the value at the upper section for each

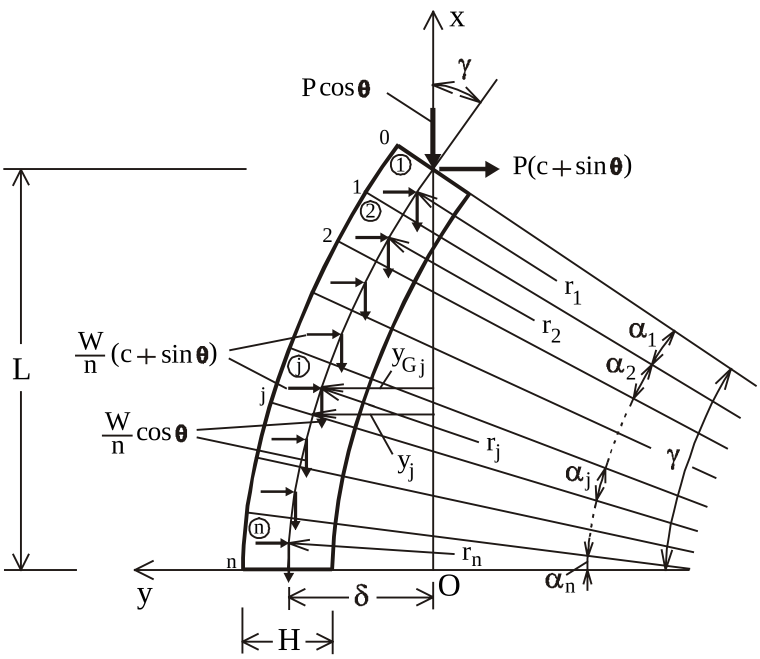

Figure 2. Deflected shape of the vertical equivalent wall strip

element. This approximation is well founded if the dimensionless length of the elements (the discretization parameter) $\xi = l/H = L/nH$ is small enough, i.e. the number n is sufficiently high. The system of coordinates $O(x,y)$ is assumed to have its x-axis through the centroid of the top cross section.

The numerical model can be used to deduce whole c-δ curve and hence $c_{max.}$ for any tilted masonry wall, using dimensionless parameters, as explained later.

Using the symbols in Figure 2, the coordinate y of the jth cross section can be written as

$$y_j = y_{j-1} + r_j \left[\cos\left(\gamma - \sum_{i=1}^{j} \alpha_i \right) - \cos\left(\gamma - \sum_{i=1}^{j-1} \alpha_i \right) \right] \quad (j = 1, 2, ..., n) \tag{1}$$

where γ = rotation of the top cross section, r_j = radius of curvature of the jth element and $\alpha_j = l/r_j$ angle related to it in the discretized model.

Expanding the cosine function in the Taylor's series and retaining the first three terms, (1), in dimensionless form, becomes

$$\frac{y_j}{H} = \frac{y_{j-1}}{H} + \xi\gamma + \frac{1}{2}\xi^2 \phi_j H - \sum_{i=1}^{j} \xi^2 \phi_i H \quad (j = 1, 2, ..., n) \tag{2}$$

where $\phi_i = 1/r_i$ = curvature of the ith element.

Since $y_0 = 0$, assuming ξ to be known, the deformed shape of the wall strip, consistent with the top rotation γ, can be obtained by using (2) recursively, starting from the index $j = 1$ [2,3]. This can be done, if the curvatures of all the elements over the cross section in consideration each time are also known.

Assuming no-tension material with linear stress-strain law in compression, the curvature of an element depends on whether its cross section is uncracked or partially cracked. By imposing the equilibrium of the jth cross section for these two different conditions, for the $(j+1)$th element the following expression of dimensionless curvature can be deduced:

$$\phi_{j+1} H = \frac{N_j}{EBH} \lambda_j \tag{3}$$

where $$\lambda_j = \begin{cases} 12\dfrac{e_j}{H} & 0 \le \dfrac{e_j}{H} \le \dfrac{1}{6} \\ \dfrac{2}{9\left(\dfrac{1}{2} - \dfrac{e_j}{H}\right)^2} & \dfrac{1}{6} \le \dfrac{e_j}{H} < \dfrac{1}{2} \end{cases} \quad (j = 0, 1, 2, ..., n\text{-}1) \tag{4}$$

E = modulus of elasticity in compression; B = width of the cross section; and N_j = resultant compressive force acting on the jth cross section with eccentricity e_j.

The resultant compressive force and bending moment acting on the jth cross section are expressed, respectively, by

$$N_j = \left[P + j \frac{W}{n} \right] \cos\theta \tag{5a}$$

$$M_j = \left[P y_j + \frac{W}{n} \sum_{i=1}^{j} \left(y_j - y_{Gi} \right) \right] \cos\theta + \left(c + \sin\theta \right) \frac{L}{n} \left[jP + \frac{W}{n} \sum_{i=1}^{j} \left(i - \frac{1}{2} \right) \right] \tag{5b}$$

where θ = angle of tilt of the wall from the vertical, Figure 1(a). Considering that $\sum_{i=1}^{j} i = j(j+1)/2$, the ratio between the quantities on the right-hand side of (5b) and (5a) provides the normalized eccentricity in the form

$$\frac{e_j}{H} = \frac{y_j}{H} - \frac{1}{n\left(P/W\right) + j} \sum_{i=1}^{j} \frac{y_{Gi}}{H} + \left(c \sec\theta + \tan\theta \right) \xi j \left[1 - \frac{1}{2} \frac{j}{n\left(P/W\right) + j} \right] \tag{6}$$

(j = 0, 1, ..., n)

which for the top cross section (j = 0), since $\sum_{i=1}^{0} y_{Gi}/H = 0$, gives $e_0/H = y_0/H = 0$.

The coordinate y of the center of gravity of the jth element, Figure 2, can be expressed by the same procedure as for the coordinate y_j of the centroid of the jth cross section. In the dimensionless form one obtains

$$\frac{y_{Gj}}{H} = \frac{y_{j-1}}{H} + \frac{1}{2} \xi\gamma + \frac{3}{8} \xi^2 \phi_j H - \frac{1}{2} \sum_{i=1}^{j} \xi^2 \phi_i H \qquad (j = 1, 2, ..., n) \tag{7}$$

Introducing (5a) into (3), the dimensionless curvature of the (j+1)th element can be written

$$\phi_{j+1} H = \frac{\rho H}{E} \xi \left(n \frac{P}{W} + j \right) \lambda_j \cos\theta \qquad (j = 0, 1, 2, ..., n\text{-}1) \tag{8}$$

where ρ = $W/(BHL)$ = weight per unit of volume of wall strip and λ_j, containing the dimensionless eccentricity of (6), is expressed by (4).

3. SOLUTION PROCEDURE

For a tilted masonry wall, the whole c-δ curve and $c_{max.}$, corresponding to assigned values of the load ratio P/W and of parameter $\rho H/E$, can be deduced by using (6), (4), (8), (2) and (7) in sequence. For this purpose, equivalent wall strip is ideally divided into sufficiently high number of elements, hence discretization parameter ξ becomes known. Assigning a trial value of the top rotation γ, using (6) and then (4) and (8) for $j = 0$, one obtains $\phi_1 H = 0$; therefore (2) provides the $y_1/H = \xi\gamma$. Then, using the equations just mentioned in the same order but for $j = 1$, one obtains y_2/H, and so on.

When the procedure stops, i.e. when the index j reaches the value n-1 in (8), the following convergence criterion is controlled which implies zero rotation at the base of the vertical equivalent wall strip:

$$\gamma = \sum_{i=1}^{n} \alpha_i = \sum_{i=1}^{n} \xi\phi_i H \tag{9}$$

It will be repeated with a decreased value of γ, if $\gamma - \sum_{i=1}^{n} \xi\phi_i H > 0$, and an increased value of γ will be utilized in the opposite case. When convergence on γ is reached, in other words, after the actual value of this rotation corresponding to an assigned value of c is iteratively determined, the deflection $y_n = \delta$ can be calculated directly from (2) [2,3].

Repeating the procedure with variation in γ and adopting small increases for this quantity, the whole curve c versus δ can be plotted and hence $c_{max.}$, that represents maximum transverse inertia force to which wall can resist, can be determined. The maximum value of γ consistent with the equilibrium of the wall corresponds to the limit condition at which the dimensionless eccentricity at the base cross section is equal to 1/2.

4. APPLICATION

The preceding procedure is now utilized to deduce the whole c-δ curve, and $c_{max.}$, for a tilted masonry wall. For the wall, Figure 3, the following data are considered: $\rho = 20$ kN/m^3; $E = 1200$ MPa; $P = 28.8$ kN. Moreover, assuming the width $B = 100$ cm, one obtains $W = \rho(BHL) = 57.6$ kN $= 2P$. Consequently, wall has $\rho H/E = 1\times10^{-5}$; $P/W = 0.5$ and slenderness ratio $L/H = 8$. Wall is ideally divided into 32 elements, so that discretization parameter ξ is 0.25.

The effect of tilting on the stability of the wall is presented in Figure 4. In this figure, the c-δ curve of intact (vertical) wall is compared to the c-δ curves of wall with $\theta = 1.0^o$, 2.0^o and 3.0^o deviations from the vertical. It is evident from this figure that, the stability decreases dramatically with the angle of tilt. The ratio between $c_{max.}(\theta = 3^o)$ and $c_{max.}$(vertical wall) is 0.026/0.078 = 0.33.

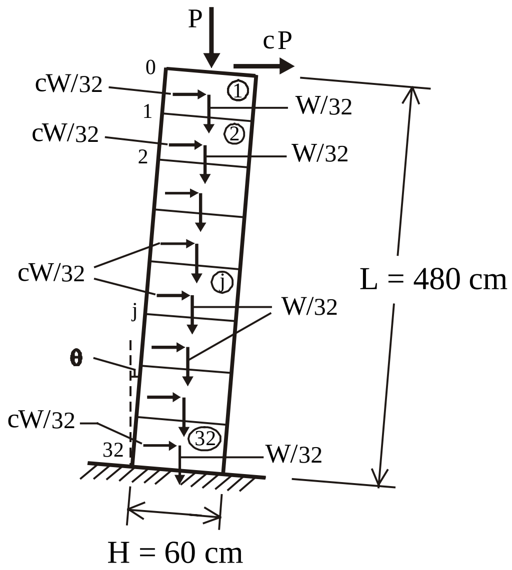

Figure 3. Tilted masonry wall considered

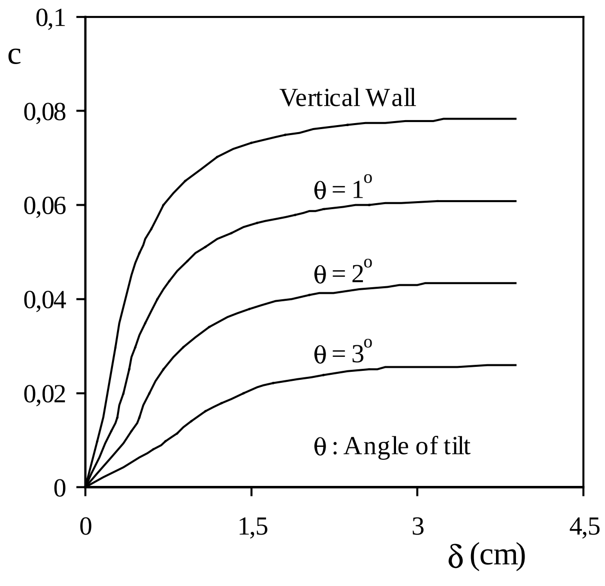

Figure 4. Effect of tilting on the stability of masonry wall considered

5. CONCLUSIONS

Many different imperfections are usually present in ruins of ancient structures, and tilting of the walls and columns is only one of them. In this work, the effect of tilting on the stability of masonry walls under seismic transverse forces was investigated and it was shown quantitatively that, Figure 4, tilting reduce the stability of the system significantly. Thus, although vertical walls seem to be stable at least to moderate earthquake ground motions, tilted ones may be quite vulnerable. The effects of several imperfections are additive and may render, especially, abandoned deteriorating ancient structures unstable during earthquakes.

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A PROPOSAL FOR BASE ISOLATION OF EDİRNEKAPI MİHRİMAH SULTAN MOSQUE

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ABSTRACT

Edirnekapı Mihrimah Sultan Mosque, one of the greatest works of Sinan, has suffered many earthquakes during its lifetime. In 17 August 1999 Kocaeli Earthquake it developed serious damages.

The analytical approaches show that an effective strengthening that will not give any harm to the original structure can not be made by using conventional techniques keeping its structural originality. A full reparation and seismic isolation, an alternative seismic retrofit technique, of the structure could lengthen the life of the structure and seems to be the unique assessment that would be radical, permanent and rational for the present.

In this study, the structure is analyzed analytically by using base isolation techniques and the results are interpreted comparatively.

1. INTRODUCTION

Historical structures are the living witnesses of the history. They are the mirrors of the social, cultural and economical life of their centuries, bridging past to the future. Preserving them, keeping their originality, should be one of the most important duties of the human beings.

In Turkey, historical buildings that reach to date are especially masonry buildings, made up of brick, stone and mortar. The heterogenity of the building materials make the analytical modelling and structural analysis of these buildings difficult. So, the analysis should be supported by experiments.

Edirnekapı Mihrimah Sultan Mosque is one of the greatest works of Sinan and has been damaged seriously in Aug. 17, 1999 Marmara Earthquake. It should be retrofitted as quickly as possible preserving its originality. Seismic isolation, an alternative approach for seismic retrofitting, could be the solution for this problem. It would lengthen the life of the strucuture and reduces the earthquake hazard effectively.

The finite element model of the mosque was made several times by using different assumptions. In 1997, Ünay has modeled the mosque, by using SAP90, finite element analysis program. In this study masonry walls are modeled as frame elements [1]. In 1999 Sayın made the finite element model of the mosque again by SAP90 using solid elements in the modelling of masonry walls. He also supported his model with ambient vibration test of the mosque made by Kandilli Observatory and Earthquake Research Institute Earthquake Engineering Department [2].

In this study, Sayın’s finite element model has been developed by refining meshes and adding some missing secondary elements. Time history analysis of the system is made by using acceleration record of 1999 Marmara Earthquake recorded in Fatih, a very near place to the Mosque. Then base isolation is adopted to the model and the time history analysis is repeated. Finally the results are interpreted comparatively.

2. EDİRNEKAPI MİHRİMAH SULTAN MOSQUE

Edirnekapı Mihrimah Sultan Complex, constructed between 1562 – 1565, is located in Hatice Sultan District in Edirnekapı, at the highest point of the inner side of the city walls. The Complex includes a mosque, a medrese, a mekteb, a türbe (tomb), a double hamam (bath) and a long row of vaulted shops in the substructure of the terrace on which the mosque and medrese is situated (Fig. 1). These shops have been torn down during the construction of the road.

Edirnekapı Mihrimah Sultan Mosque has suffered damages in the earthquakes of 1640, 1690, 1719, and 1894. In 1719 its domes collapsed. After 1894, it remained derelict for many years before the Ministry of Pious Foundations undertook its reparation. But the work was discontinued. It was not opened to public until 1956 when the mosque was extensively restored and saved [3].

2.1 Structural System of the Mosque

The 39.50 by 28.00 m prayer hall of the Edirnekapı Mihrimah Sultan Mosque is surmounted by a central dome, 20.25 m in diameter. Inside the hall, the low triple domes of 6.0 m diameter, rest on two granite columns with stalactite capitals. The central dome on four pendentives supportedby four main arches, rises on four piers which turn into weight towers above the cornice level.

The structure is symmetrical in east – west direction. But the rigidity difference between the north and south walls causes a non-symmetry in the east – west direction. Higher rigidity of the north wall than the south wall could be one of the main reasons for the high level of damage in the south wall.

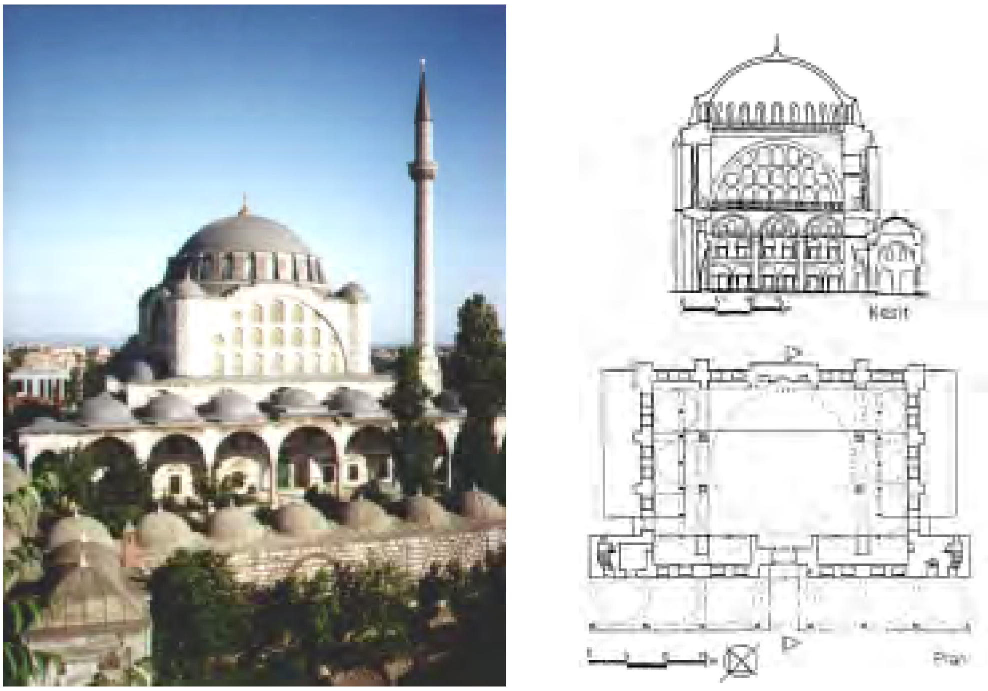
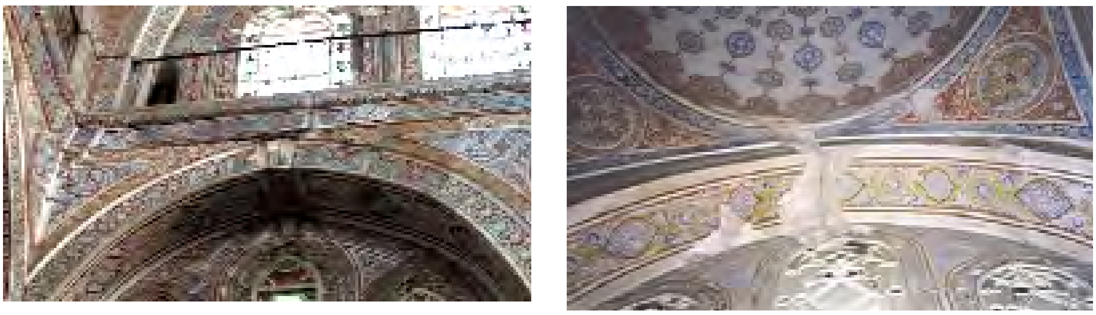


Figure 1. General view, plan and cross-section of the Mosque

2.2 Observed Damages

1. There are cracks around the key stone level of the two arches at the gallery level of the south wall. One of the arch's key stone has made a great displacement (Fig. 2.a, b)



(a) (b)

Figure 2 Damaged arches in the gallery level of the south wall

2. The main arch in south wall is damaged. Three stones of the arch had fallen down during the 1999 Marmara Earthquake (Fig. 3)
3. The south-east pendentive has a great crack on the level of the fallen stones of the main arch (Fig. 4.a)
4. The south perferated wall beneath the main arch is seriously damaged (Fig. 4.b)

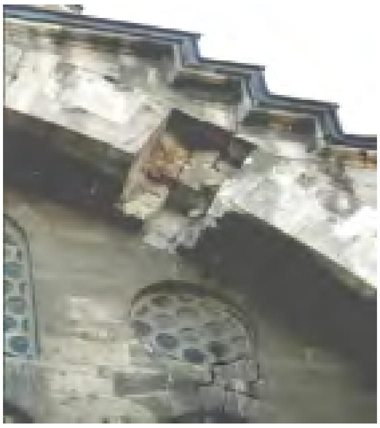
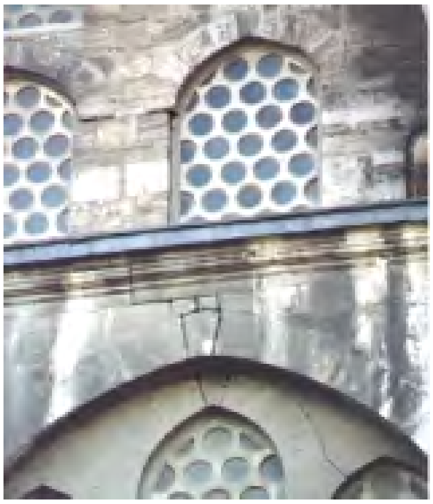


Figure 3. Main arch in the south wall

a



b

Figure 4 The south-east pendentive (a) and the south wall (b)

2.3 Soil Condition of the Site

In July 2000, soil drilling was made and soil layers were determined and samples were taken by Yertek Engineering. In September 2000, tests were made on the samples by YTÜ Civil Engineering Department Geotechnic Division. In the report of these tests and the drilling, it is stated that theere is an artificially filled soil formed of clay, sand and rock pieces, till 2.50m from the surface. Between 10.00 m and 23.50 m, there is a layer of limestone. Under this layer green colored stiff clay mixed with sand. The load bearing capacity of the soil is 175 kPa. The characteristic periods of the soil are: T_A = 0.15s and T_B = 0.60 s [4].

2.4 Material Properties Used in the Analysis

The Mosque's domes and pendentives are of brick, the walls and the piers are of stone. In this study no material test was made. The material properties are defined according to the results of the ambient vibration test made by Kandilli Observatory and Earthquake Research Institute Earthquake Engineering Department.Table 1 shows the material properties used in this analysis.

2.5 Free Vibration Analysis

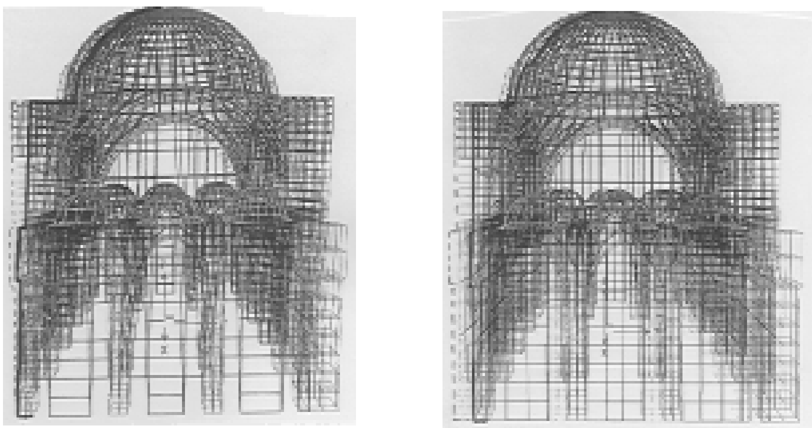
The fixed base and base isolated models are analysed. The periods of the first three modes of the structure is given in Table 2. In Fig. 5 - 7 the first three mode shapes are shown.

Table 1 Material Properties Used in the Analysis

Element Type	Modulus of Elasticity (N/mm^2)	Unit Weight per Volume (N/mm^2)	Poisson's Ratio
Solid	7000	2.19×10^{-5}	0.15
Shell (used for walls)	7000	2.19×10^{-5}	0.15
Shell (used for domes, pendentives)	3000	2.00×10^{-5}	0.15

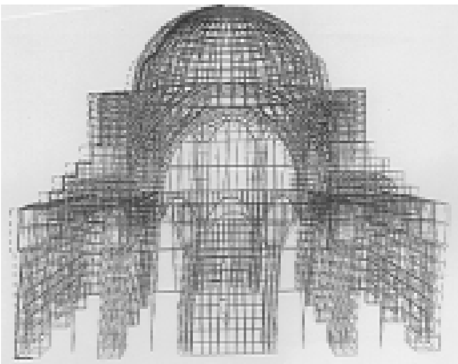
Table 2 The First Three Mode Periods of the Fixed Base and Base Isolated Models

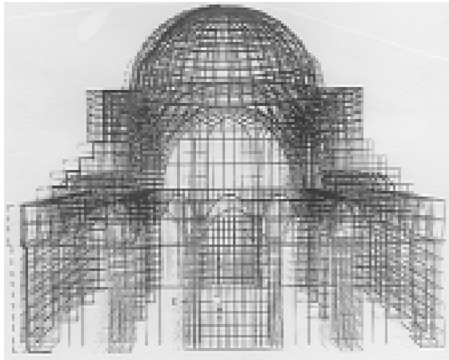
Mode	Fixed Base Period (s)	Base Isolated Peiod (s)	Main Direction
1	0.4422	2.00	North – South
2	0.4011	2.00	East – West
3	0.1702	1.86	Torsion



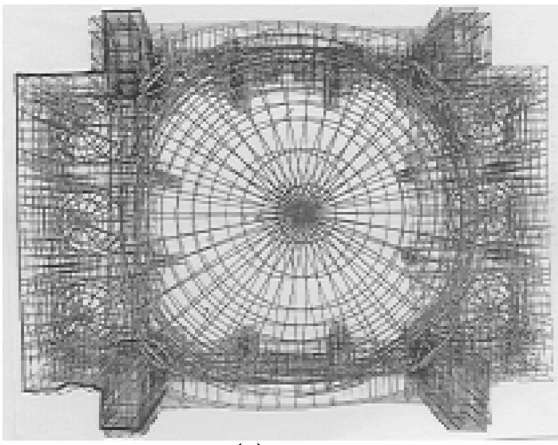
(a) (b)

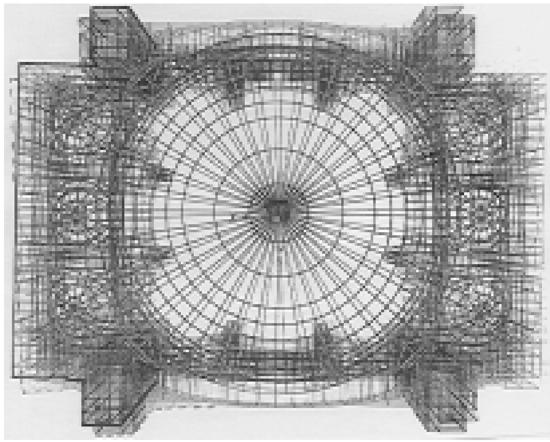
Figure 5 The First Mode Shape of Fixed Base (a) and Base Isolated (b) Structures





(a) (b)
Figure 6 The Second Mode Shape of Fixed Base (a) and Base Isolated (b) Structures

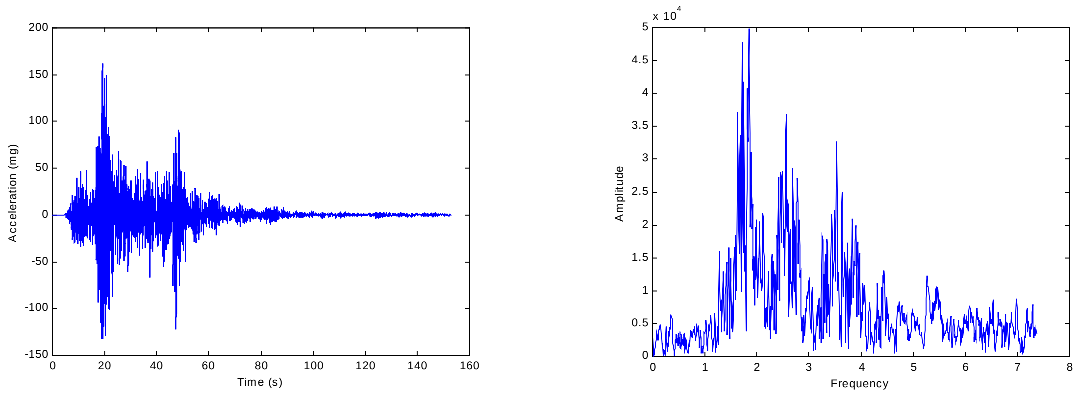




(a) (b)
Figure 7 The Third Mode Shape of Fixed Base (a) and Base Isolated (b) Structures

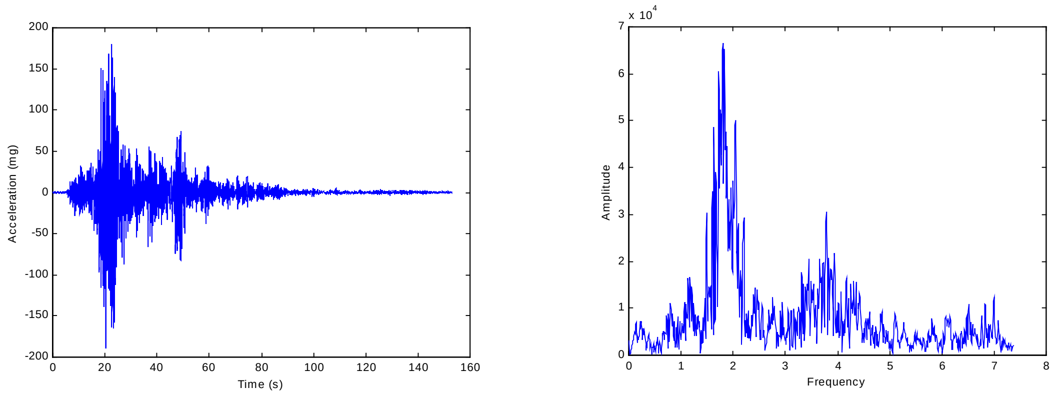
2.6 Time History Analysis

In this analysis the acceleration record of 1999 Marmara Earthquake taken from Fatih Tomb is used. Edirnekapı Mihrimah Sultan Mosque is very near to the site. For understanding the characteristic of the record, fourier transform is made and fourier amplitude spectrum is obtained by using a program made in MATLAB. In Fig. 8 and 9 the accelaration records and the fourier amplitude spactrums for these records are given. The spectral ground accelerations and approximate main periods of the record are shown in Table 3.



(a) (b)

Figure 8 Acceleration Record (a), Fourier Amplitude Spectrum (b) (East – West)



(a) (b)

Figure 9 Acceleration Record (a), Fourier Amplitude Spectrum (b) (North – South)

Table 3 Spectral Ground Accelerations and Approximate Main Periods of the Record

Direction	**Spectral Acceleration (g)**	**App. Main Period (s)**
North – South	0.189	0.55
East - West	0.162	0.54

The forced vibration periods of the structure is given in Table 4. In tables 5 and 6 the maximum acceleration and displacement values at some joints of fixed base and base isolated structures are given.

Table 4 Forced Vibration Mode Periods

Mode	Period (s)
1	0.4823
2	0.4319
3	0.2765

Table 5 Maximum Displacement Values for Fixed Base and Base Isolated Structures

Joint No.	Displacement (cm)			
	Fixed Base		Base Isolated	
	E-W	N-S	E-W	N-S
2012 (Base level)	-	-	2.813	2.292
328 (Top point of the main arch in the south wall)	3.034	7.647	2.936	2.612
1328 (Top point of the main arch in the north wall)	3.707	7.254	3.066	2.675
478 (Top point of the main arch in the west wall)	4.033	5.901	3.030	2.537
1478 (Top point of the main arch in the east wall)	4.148	5.909	3.011	2.573
6008 (Top point of the main dome)	3.77	6.18	3.015	2.560

Table 6 Maximum Acceleration Values for Fixed Base and Base Isolated Structures

Joint No.	Acceleration (g)			
	Fixed Base		Base Isolated	
	E-W	N-S	E-W	N-S
2012 (Base level)	-	-	0.164	0.192
328 (Top point of the main arch in the south wall)	0.653	1.14	0.171	0.218
1328 (Top point of the main arch in the north wall)	0.823	1.166	0.171	0.216
478 (Top point of the main arch in the west wall)	0.929	0.93	0.172	0.211
1478 (Top point of the main arch in the east wall)	0.911	0.914	0.173	0.211
6008 (Top point of the main dome)	0.808	0.956	0.172	0.213

3. CONCLUSIONS

According to the time history analysis results, the following conlusions can be provided:

1. The top point of the main arch of the south wall of the building is making 7.64 cm displacement in the north-south direction. This high level of the displacement could be the result of the small difference between the main period of the acceleration record and the building's first mode period.Stones' falling down from the main arch in the south wall during the earthquake seems to prove the results obtained from the analysis.
2. The high values of spectral ground acceleration in Fatih, which is approximately 100 km far from the epicenter, could be because of the soil condition.
3. There is approximately 0.5 cm difference between the displacements of south and north walls. This shows the rigidity difference between the walls and the high damage of south wall while the level of damage is low in the north wall.
4. In these circumstances retrofit of the structure necessiates a change in the dynamic characteristics of the building. This could be obtained by including new elements to the structure, but this can give harm to the originality of it.
5. Analysis shows that the displacement difference between the base and top level of the structure and the acceleration values at the joints is reduced significantly by using base isolation. Base isolation would give no harm to the super structure and if economical spects are not considered, seems to be the best solution.

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DYNAMIC RESPONSE OF CHURCH STEEPLES

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ABSTRACT

The paper deals with the steeples of three-lobed orthodox churches. They are typical components of ecclesiastic architecture and structurally integrated in church bodies. Under seismic actions some steeples are damaged and even dislocated. The paper presents potential mechanisms of damaging, some results of numerical analysis and practical methods to prevent or repair damage in church steeples.

1. INTRODUCTION

Orthodox churches characterised by a three-lobed plan are endowed with one or more steeples symmetrically located with respect to church longitudinal axes. Arches or vaults support the steeples and bearing columns or walls there are not needed any longer. The aspect ratio of steeples, D-diameter/H-height, is usually between 1:2 and 1:3.25. As an aesthetic parameter it defines steeple slenderness in the ecclesiastic architecture. However, in seismic areas steeples do not last long. Under rather weak shakes they are damaged or even cut, dislocated and ultimately destroyed. This is the reason why the new steeples, that replaced the original ones in many three-lobed plan churches, were made shorter or built by wooden frames covered with metallic sheets. Such alterations affect the aesthetics of orthodox churches [3].

In previous studies it was shown that the three-lobed plan of orthodox churches derives from the Greek straight cross but was strongly influenced by the Roman architecture. Starting from 1512 the seismic protection of three-lobed plan was enhanced by enlarging the narthex [5,6,7]. However, their steeples remained vulnerable. The method of checking their shears resistance based on Bredt’s theory hardly changed the situation of existing steeples. [4]. By theoretical and numerical analysis the paper emphasises some potential mechanisms of damaging, and suggests a technique of prevention or repair damages in steeples [9,10,11,12].

2. A LESSON FROM MECHANICS

Assume the space trajectory of a typical point during an earthquake, like the centre of gravity of a church building or only that of its steeple, is defined by the function

$$T = T\left(\vec{M};\vec{\tau},\vec{n},\vec{\beta}\right), \tag{1}$$

where $\vec{M}$ is the point position vector, while $\vec{\tau},\vec{n},\vec{\beta}$ represent a right-handed triple of orthogonal unit vectors: tangent, principal normal and bi-normal vector, respectively. They satisfying the conditions

$$\vec{\tau}x\vec{n} = \vec{\beta} \text{ and } \vec{\tau}\vec{n} = 0. \tag{2,3}$$

The four vectors, in function of trajectory length s, are related through the following relations

$$\frac{d\vec{M}}{ds} = \vec{\tau};\quad \frac{\mathrm{d}^2\vec{M}}{ds^2} = k\vec{n};\quad \frac{\mathrm{d}^3\vec{M}}{ds^3} = -k^2\vec{\tau} + \frac{dk}{ds}\vec{n} + k\kappa\vec{\beta}, \tag{4,5,6}$$

and

$$\frac{d\vec{\tau}}{ds} = k\vec{n};\quad \frac{\mathrm{d}\vec{n}}{\mathrm{ds}} = -k\vec{\tau} + \kappa\vec{\beta};\quad \frac{\mathrm{d}\vec{\beta}}{\mathrm{ds}} = -k\vec{n}, \tag{7,8,9}$$

where k is the curvature of trajectory,

$$k = \frac{1}{\rho} = \sqrt{\left(\frac{d^2\vec{M}}{ds^2}\right)^2}, \tag{10}$$

while κ is its torsion,

$$\kappa = \frac{1}{r} = \frac{\frac{d\vec{M}}{ds}\,\frac{d^2\vec{M}}{ds^2}\,\frac{d^3\vec{M}}{ds^3}}{\left(\frac{d^2\vec{M}}{ds^2}\right)^2}. \tag{11}$$

The velocity vector of the considered point along its trajectory is

$$\frac{d\vec{M}}{dt} = \vec{v} = v\vec{\tau}, \tag{12}$$

where

$$v = \frac{ds}{dt}. \tag{13}$$

The second derivative of vector radius $\vec{M}$, by also considering (7), gives

$$\frac{d^2\vec{M}}{dt^2} = \frac{d\vec{v}}{dt} = \frac{dv}{dt}\vec{\tau} + v\frac{d\vec{\tau}}{ds}\frac{ds}{dt} = \frac{dv}{dt}\vec{\tau} + kv^2\vec{n}. \tag{14}$$

The third derivative of the same vector gives

$$\frac{d^3\vec{M}}{dt^3}=\frac{d^2\vec{v}}{dt^2}=\frac{d^2v}{dt^2}\vec{\tau}+\frac{dv}{dt}\frac{d\vec{\tau}}{ds}\frac{ds}{dt}+\frac{dk}{ds}\frac{ds}{dt}v^2\vec{n}+k2v\frac{dv}{dt}\vec{n}+kv^2\frac{d\vec{n}}{ds}\frac{ds}{dt}.$$

By successively considering (7), (8) and (13), and by collecting the terms multiplied by the same unit vectors one finally obtains the variation in time of the acceleration vector

$$\frac{d^3\vec{M}}{dt^3}=\left(\frac{d^2v}{dt^2}-k^2v^3\right)\vec{\tau}+\left(3kv\frac{dv}{dt}+v^3\frac{dk}{ds}\right)\vec{n}+k\kappa v^3\vec{\beta}. \qquad (15)$$

By multiplying it by the mass of moving point one finds the expression of jerk $\vec{J}$, the sudden and violent force on time unit, that keeps the mobile on its space trajectory during an earthquake [1]. Its first component

$$J_\tau=m\left(\frac{d^2v}{dt^2}-k^2v^3\right), \qquad (16)$$

has the function of translation or displacement along the direction of tangent, the second one

$$J_\nu=m\left(3kv\frac{dv}{dt}+v^3\frac{dk}{ds}\right), \qquad (17)$$

controls the rotation or bending in the osculating plane, while the third one

$$J_\beta=mk\kappa v^3, \qquad (18)$$

governs the rotation or torsion in the rectifying plane of the trajectory [2].

By setting equal to zero expression (16) one finds the non-linear equation for geometric and mechanic parameters of motion

$$\frac{d^2v}{dt^2}=k^2v^3 \qquad (19)$$

that deletes jerk's component of translation.

Similarly, from (17) one finds the equation

$$\frac{dv}{dt}=-\frac{v^2}{3k}\frac{dk}{ds}, \qquad (20)$$

and, for two consecutive instants 1 and 2, its solution

$$v_2=v_1\sqrt[3]{\frac{k_1}{k_2}} \qquad (21)$$

gives the value of velocity for which jerk's component of rotation disappears.

Finally, it's worth noticing that as long as the trajectory is spatial jerk's torsion component never disappears. Moreover, since it varies with the cubic power of velocity, for small radii of curvature and torsion it could attain high values. Therefore in the case of earthquakes, torsion effects cannot be avoided and they are extremely dangerous, mainly for rigid buildings. Most of damages to churches are due to the torsion component of jerk. In the particular case of Bucharest two typical parameters of ground motion have been identified: 0.23g, 27 m/s and 18 cm, generated by the source located at 90 km depth, and 0.52g, 105 cm/s and 42 cm, generated by the source located at 150 km depth [8]. Assuming in (18) $k = \kappa = max.displacement$ one finds for the two limits of PJT (peak jerk torsion)

$$J_\beta = (0.062 - 0.668) Weight/s. \tag{22}$$

It is expected that these rather high values obtained theoretically to be checked and accordingly adjusted by both numerical analysis and lab tests.

3. NUMERICAL ANALYSIS

The three-lobed church of Caldarusani Monastery is located on the NW Shore of Caldarusani Lake, in Snagov Plane, not far from Bucharest. It was the most important ecclesiastic monument built under the reign of Prince Matei Basarab between 1637 and 1638 (Fig. 1). During its long service earthquakes damaged the church many times. The most affected parts were its three steeples, the main one with the function of Pantokrator and the twin ones over the narthex.

Figure 1 Cadarusani Church

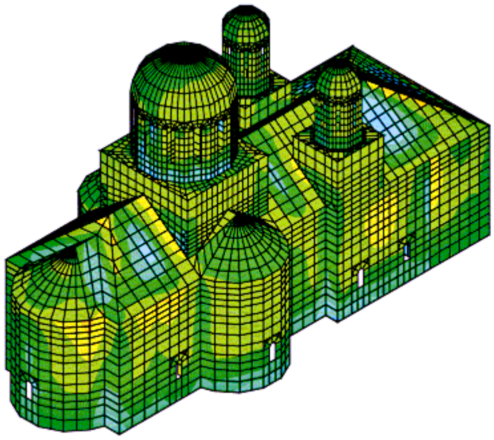

Figure 2 Model for static analysis

In order to find out the mechanism of damaging during earthquakes, the church and its steeples were modelled by using both 3D finite elements and plate elements (Fig.2). The numerical investigation was carried out in the framework of INCO Copernicus Project IC15-CT97-0208 *INRESQUAKEPACT*. For both static and dynamic analysis Drucker-Prager's yield condition was adopted. In the case of dynamic analysis, soil-structure interaction was obviously considered. Special attention was also given to the dynamic response of church steeples (Fig. 3). For this purpose the critical areas in steeple structures have been identified. It was found out that most of them are located on or near steeple bases, where cross sections suddenly change (Fig. 4). The most important dynamic parameter was the response acceleration because it characterises the inertia forces developing during earthquakes. With the aid of the same parameter it was observed if the induced forces are amplified or attenuated by steeple structures. One could also examine what happens in the other church structural members at the same time. A similar comparative analysis was developed for the dynamic state of stresses. Resistance phenomena are strongly non-linear, and consequently, beyond of any human intuition. Finally, the time-history analysis of the strains in the same sections allows one to assess the existing reserves of ductility and how they could be enhanced by structural interventions.

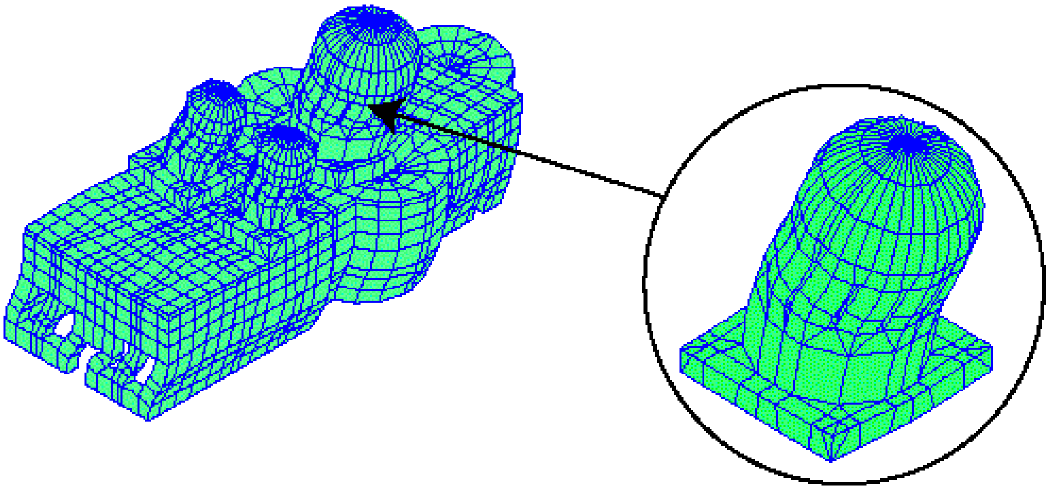

Figure 3 Model for non-linear dynamic analysis using Drucker-Prager's condition

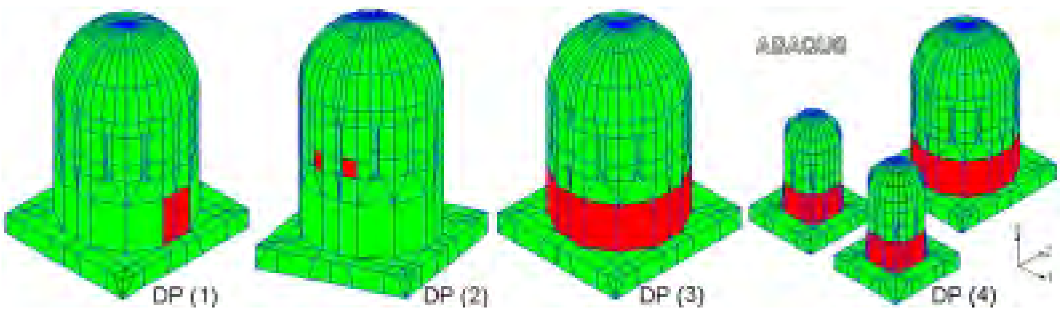

Figure 4 Steeple models considered in dynamic non-linear analysis

The results of non-linear analysis have shown that the main Pantokrator steeple had the worst dynamic response to El Centro'40 earthquake. Indeed, its response acceleration is almost four times larger than those of small steeples in the transverse direction 1 and three times larger in the longitudinal direction 2 (Figs. 5, 6). Even in the vertical direction 3 the dynamic response of the main steeple was larger than of any other church structural members. Surprisingly, the best dynamic behaviour was recorded for arches and partition walls and it can be explained by the reduced mass of these structural components. The result is comforting mainly when the preservation of three-lobed is considered. As it was expected the dynamic response of external walls should be carefully examined. When possible, e.g. when new churches are designed or existing ones are restored, geometric equilibration of masses is recommending by adopting adequate shapes. The lesson of EQ Engineering given five centuries ago by Manole Master with his outstanding Church in Curtea de Arges is still of current interest.

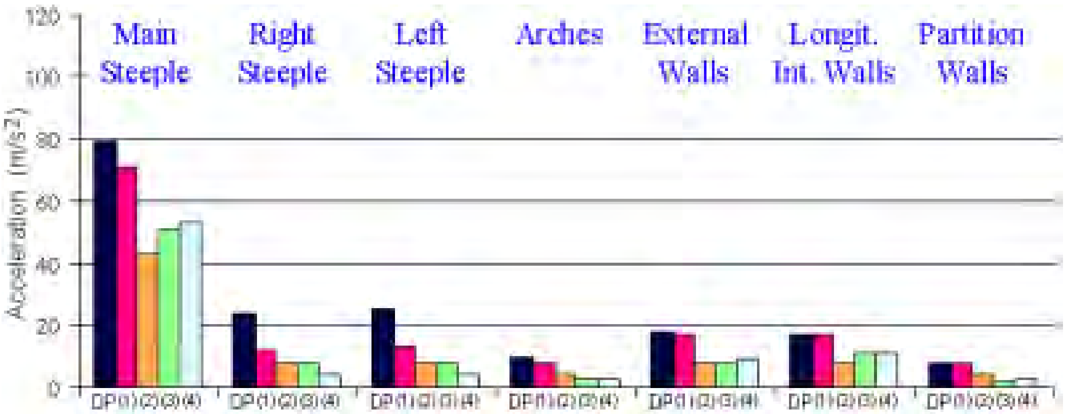

Figure 5 Acceleration a_1 in church structural elements sets for the time t = 2.72 s

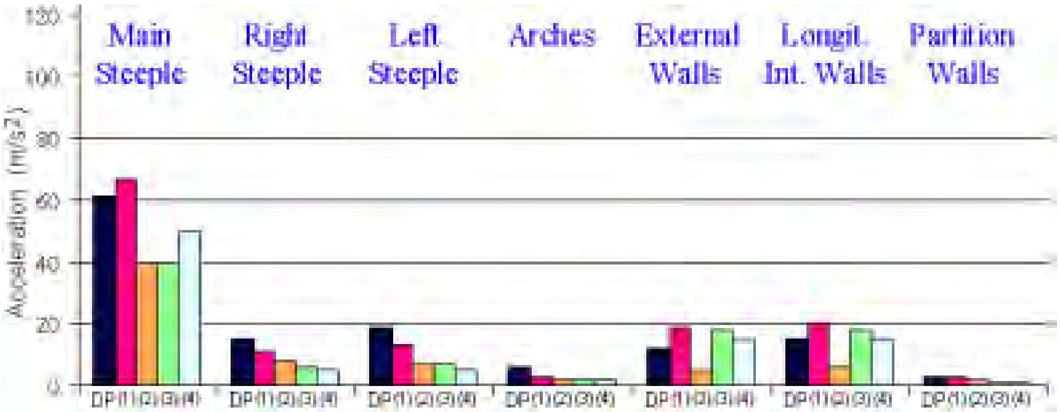

Figure 6 Acceleration a_2 in church structural elements sets for the time t = 2.72 s

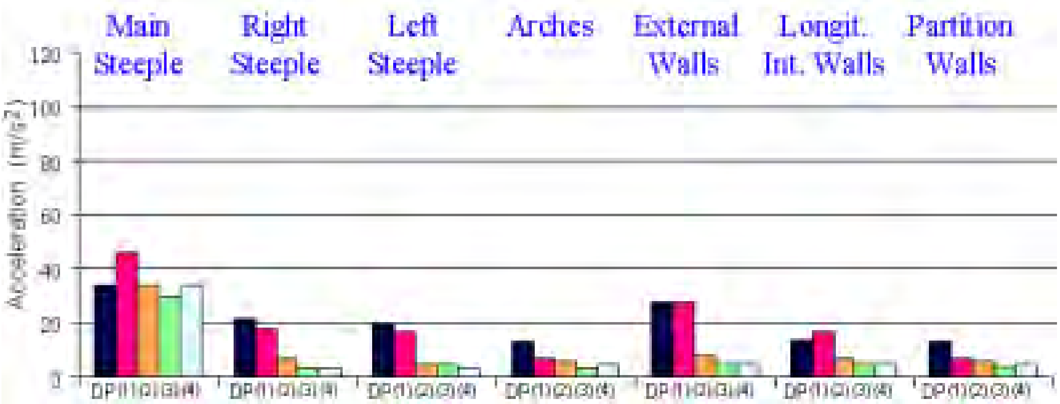

Figure 7 Acceleration a_3 in church structural elements sets for the time t = 2.72 s

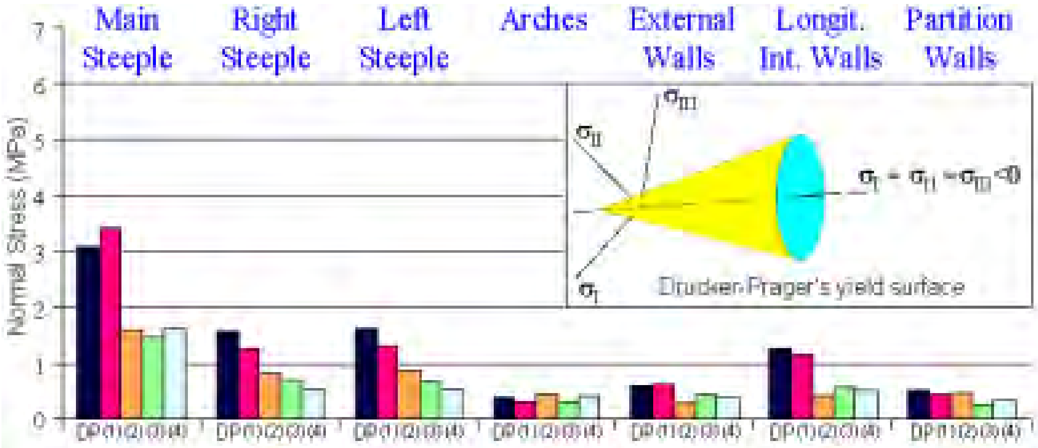

Figure 8 Normal stress σ_{11} in church structural elements sets for the time t = 2.72 s

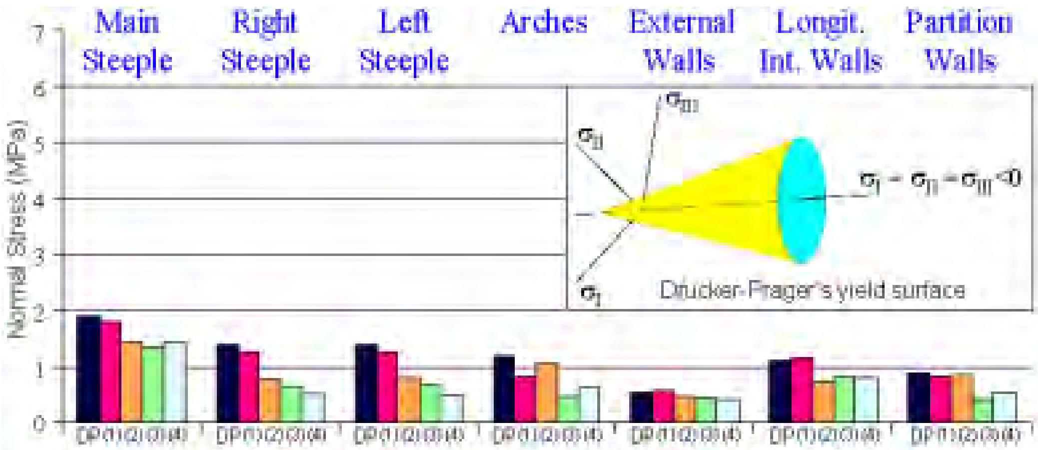

Figure 9 Normal stress σ_{22} in church structural elements sets for the time t = 2.72 s

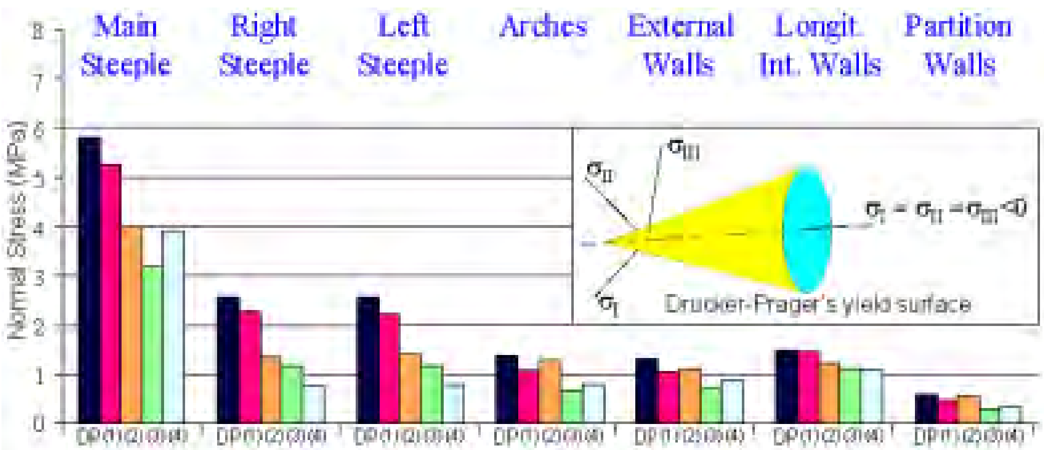

Figure 10 Normal stress σ_{33} in church structural elements sets for the time t = 2.72

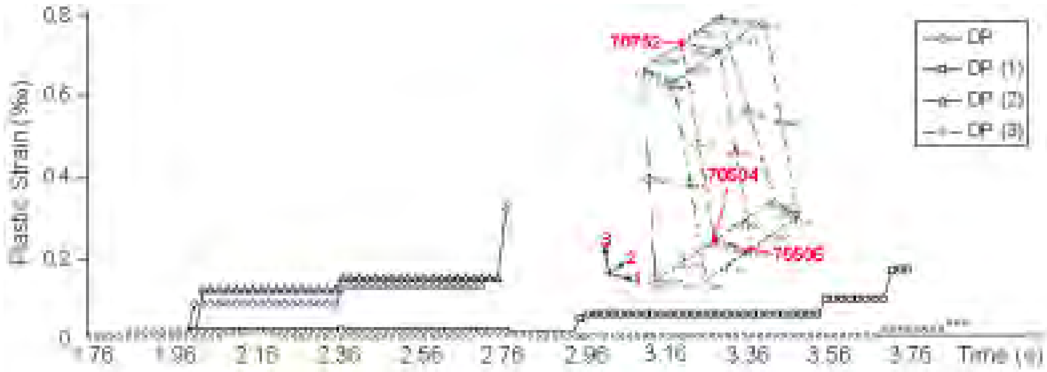

Figure 11 Plastic vertical strains ε_{33} in the node 70504 of main steeple

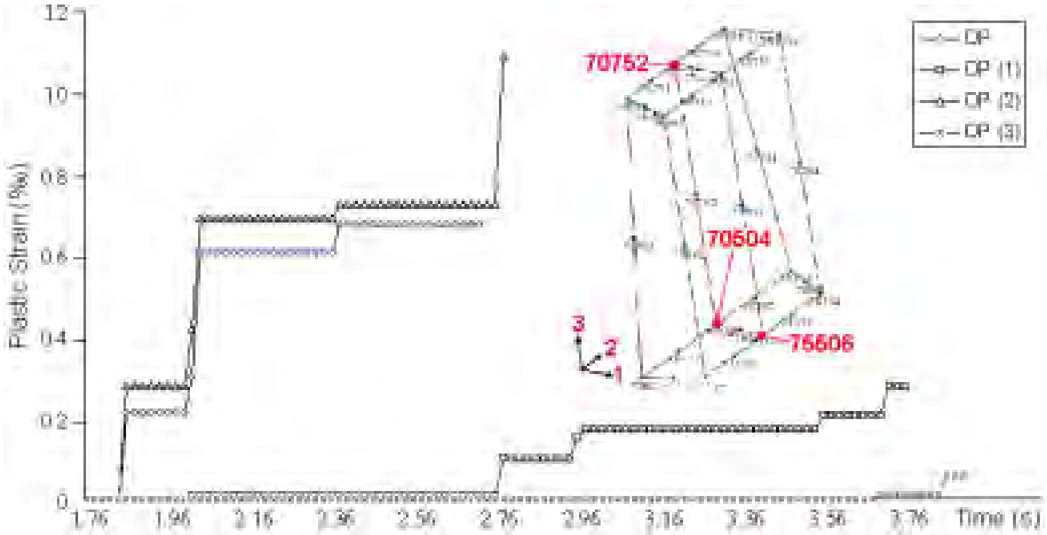

Figure 12 Plastic vertical strains ε_{33} in the node 75506 of main steeple

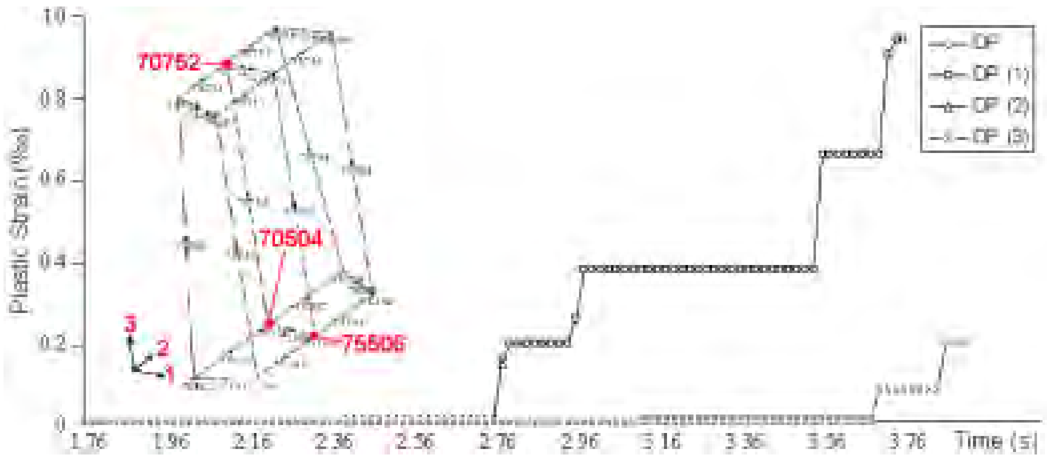

Figure 13 Plastic vertical strains ε_{33} in the node 70752 of main steeple

Normal stresses in the non-linear analysis of dynamic response are again much larger in the main steeple than in any other church structural members (Figs. 8, 9, 10). Naturally, for this type of stresses the vertical direction 3 is the most dangerous one, because axial tensions and compressions are successively developing. It is followed by the lateral direction 2 in which the church stiffness in bending is minimum. In the longitudinal direction 1 almost all structural members display the same moderate dynamic response.

The time history diagrams of vertical plastic strains obtained by numerical non-linear analysis for the same El Centro'40 input are essentially discrete (Fig.11, 12, 13). The strains at the three points chosen for analysis are developing at different instants and attain different values, without any connection between them. They are typical for a ductile behaviour occurring in the original masonry of the main steeple, before strengthening with polymer grids. By plastic deformation the stored elastic energy is dissipated avoiding high concentrations of stresses. The mechanism is successively repeating in different sections until the entire capacity of dissipation is spent. Then cracks followed by dislocations occur.

4. DAMAGE PREVENTION OR REPAIR

Seismic vulnerability of masonry in church steeples is due to its brittleness. It has not enough capacity to dissipate the induced energy and under seismic actions it is easily subjected to cracks or is even crushed, dislocated and expelled. Originally, masonry was not brittle but ductile. Indeed, solid bricks made of burnt clay were porous and elastic, while the lime mortar had essentially plastic qualities. Two inventions of the Industrial Revolution changed the millennial masonry: 1) cement and 2) cored bricks. Cement mortar is not ductile any longer but brittle, while hollowed ceramic bricks are not only extremely brittle but also responsible for stress concentrations on the reduced areas of contact between bricks and mortar layers. The question is shall we return to the original masonry? The answer is definitely no. History never returns. After years of research it was found that polymer grids could solve the problem.

There are three criteria for choosing polymer grids as reinforcement for masonry: 1) high tensile strength at low strains, 2) controlled long-term deformations and 3) integrated solid joints. The reinforcing approach with synthetic grids essentially differs from the technique known for steel bars. Polymer grids are firmly fixed in mortar by interlocking of their joints. The mechanism of stress transfer from mortar to grids is discontinuous and takes place only in the solid joints through normal stresses σ, without any contribution of the tangential ones τ. Only tensile forces are transferred from mortar to grids. This specific mechanism was demonstrated by a pullout test on a grid inserted in sintered glass with glycerine and viewed under polarised light.

Polymer grids show great potential for preventing or repair damage in church steeples. Similarly, they are effective for reinforcing and confining masonry with.

This work involves two specific techniques: 1) inserting the grids in the horizontal layers of mortar between bricks; 2) confining the masonry of steeple structures with the same reinforced plaster. In all cases, synthetic reinforcement compensates for masonry's lack of ductility and enhances its natural strength capacity (Fig.14).

Figure 14 Horizontal and vertical reinforcement

The first technique improves load transfer capacity between the masonry units, since the reinforcement prevents horizontal expansion of mortar. It is not necessary to lay the grids in all mortar beds, but only in some of them at vertical distances between 20 cm and 60 cm. The joints of grids are obtained by superposition without any specific devices. Confinement with reinforced plaster improves both compression and shears resistance and is most efficient when combined with the reinforcement in horizontal layers. This type of reinforcement acts in a three-dimensional sense and can be used to increase the bearing capacity of steeple structures several times.

Reinforcing and/or confining endow masonry with exactly those strength and ductility which are able to reduce the seismic vulnerability of church steeples. Both laboratory tests and theoretical models validate the proposed composite masonry, while the existing database supports any conceptual design. The method is easily applied and financially attractive. Grid supplementary cost of reinforced masonry is about 0.6%, while that of confined masonry only increases up to 6%.

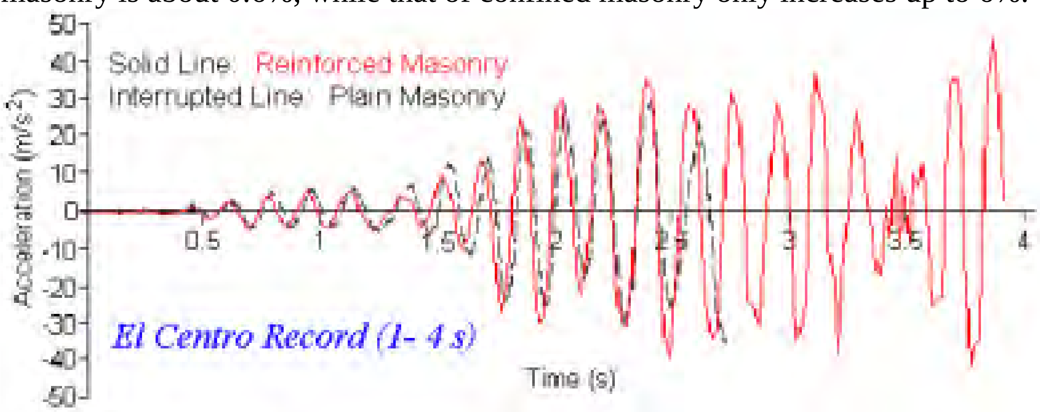

Figure 15 Dynamic response of main steeple before and after reinforcing

As an application the dynamic behaviour of the main steeple of Caldarusani Church was comparatively analysed in two alternatives: 1) the actual state, with plain masonry and 2) slightly strengthened by confining with polymer grids but for sake of simplicity only around its base (Figs. 3, 4). In the numerical analysis for both cases Drucker-Prager's yield condition with perfect plasticity was assumed. The steeple model was then successively submitted to an input according to El Centro'40 record with a peak value of 0.5g. This value is 2.5 times larger than that provided for design by Romanian Code P100-92. The dynamic response of the actual steeple lasted only 2.72 seconds when its plain masonry suddenly failed. To the same strong input the non-linear model of the steeple with confined base lasted 4 seconds, a time with 48% longer (Fig. 15). Even if it was not a final solution of strengthening the result shows the remarkable contribution of polymer grids to mitigate the dynamic response of church steeples under seismic actions.

5. CONCLUSIONS

1) The sudden and violent force developing during earthquakes called jerk might bring new explanations on the dynamic response of church steeples. The jerk torsion component enforces very high shears stresses. They simply cut steeple columns along two perfect horizontal interfaces, at their tops and bottoms.
2) The numerical analysis brought extremely valuable information about the dynamic response of steeples in three lobed churches. Beside response accelerations, the values and distribution of stresses and strains in steeple critical areas were obtained. The most important result consists in discovering the phenomenon of dynamic amplification of the induced inputs. It appears like an effect of ***"whip cracking",*** explaining the high seismic vulnerability of church steeples.
3) The method proposed for prevention or repair damages in steeples is based on polymer grids. It has proved to be very practical and cost effective. Beside **"*masonry ductilisation*"** the most important effect of synthetic reinforcement consists in a convenient distribution of stresses. In this way stress concentrations, always responsible for cracks and damage in masonry, are prevented. Masonry reinforced and/or confined with polymer grids fulfil the conditions requested to genuine composite materials. They are mainly based on the Saint Venant's Principle of geometric continuity.

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THE DYNAMIC BEHAVIOUR OF THE BASILICA S.MARIA DI COLLEMAGGIO

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ABSTRACT

The dynamic behaviour of the Basilica S.Maria di Collemaggio was established in order to assess the efficiency of some small-scale restoration to repair minor damage, due to recent earthquakes, and to moderately strengthen the structural elements. As a preliminary step a parametric analysis was performed by finite element models to predict and frame the response of the church. Experimental tests allowed the determining of the modal parameters and the efficiency of structural elements before the works. In this paper numerical and experimental studies are presented and discussed.

1. INTRODUCTION

The Basilica S.Maria di Collemaggio is one of the most attractive churches in Centre Italy (Figure 1). It dates from the XV century. Some small-scale restoration are underway in order to repair minor damage due to recent earthquakes and to moderately strengthen the structural elements.

The Basilica has a nave and two side aisles. The dimension of the nave is 61m in length and 11.3m in width; its height reaches 18.25m. The two side aisles are 7.8 and 8m in width; two external walls both 12.5m high delimit them.

Seven columns, not evenly distanced, on each side separate the nave and two side aisles. The columns are about 5.25m high; a layer of well-laid stone, made of a calcareous material arranged irregularly in a poor quality mortar, encloses their core; the transverse section, approximately circular, is on average 1.00 m in diameter.

The thickness of masonry varies from 0.95 m to 1.05 in the external walls; it is 0.9m in the two walls of the nave, over the columns. The four walls are connected on one side to the facade of the Basilica and, on the other side, to the transept. The facade is joined to a thick octagonal tower on the right corner; another masonry

Figure 1 A view of Basilica S.Maria of Collemaggio

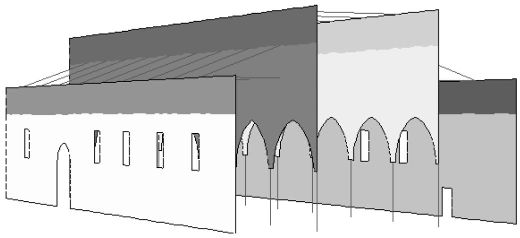


Figure 2 A sketch of four longitudinal walls in the numerical models

building is adjacent to a part of the wall, about 40% of it, behind the tower. The wooden roof is supported from trusses placed in a cross-sectional direction to the walls.

A numerical and experimental study has permitted to characterise the dynamic behaviour of the Basilica. A parametric analysis was performed by FE models to predict and frame the response of the church. Experimental tests allowed the determining of the modal parameters and the efficiency of structural elements. The results of dynamic tests can be used to define a predictive model as well as, in this case, to validate and update finite element models of the structure. Furthermore, they can give information on the overall behaviour [5] that often depends, for structures like this, not only on the mechanical properties of the materials. More in general, their variations systematically observed in time can allow the evidencing of those changes related to structural integrity.

Preliminary numerical analyses were carried out on the basis of several assumptions regarding: 1 mechanical parameters of masonry, 2. timber trusses of the roof, 3. restraints in walls and columns, 4. links among structural components. They permitted a reasonable prediction of the behaviour of the church to be made.

Afterwards the Basilica was excited at a low level by an instrumented hammer and a mechanical vibration exciter (vibrodyne). Six accelerometers registered responses. Several tests have been carried out, with different positions of the instruments and impact locations, in order to excite and to measure as many modes as possible. The vibrodyne was located on the top of a lateral wall. The frequency responses were directly measured around the first two modes; these are the most important ones that describe the dynamic response of the church. Experimental data have been used to identify natural frequencies, modal displacements and damping factors. Other tests are planned at the conclusion of retrofitting.

2. FINITE ELEMENT MODELS

A standard finite element model has been developed to evaluate the sensitivity of the modal shapes and frequencies to a set of mechanical parameters describing the masonry walls. The mesh of the model has been adapted in order to respect all possible geometric characteristics of the church, such as the openings in the walls and the imperfect parallelism between the walls of the nave.

Only the four main longitudinal walls (Figure 2) have been considered in the model using plate-shell and beam elements. Even if the facade and transept are stiff enough to give approximately null displacements in their plane, they have been modelled through the tuning of opportune flexible restraints at the extremity of the longitudinal walls.

Standard triangular plate-shell elements (6673 in number) have been used to reproduce, as accurately as possible, the real geometry of masonry walls of the Basilica. In the optimised mesh an element average size of 100 cm has been considered. Plate-shell elements take into account the real thickness of the masonry walls.

Columns have been modelled by beam elements, while truss elements have been used for the wooden elements of the roof.

Structure-soil interaction has been neglected, therefore the outer walls and the columns, are considered fixed to the soil. The wooden beams of the roof have been considered with and without any connections to the masonry. These two limit cases provide the influence on the frequencies of the horizontal interaction between the walls, at roof level.

Particular attention has been given to the non-homogeneous distribution of the physical and mechanical masonry characteristics. The four walls have been modelled equally through two horizontal bands with different elastic modulus and mass density. In each band the material is assumed to have linear elastic behaviour, and homogeneous and isotropic characteristics.

Table 1 Finite element model characteristics

	Element type	**Material**	***E* [MPa]**	**ν**	***M* [Kg/m^3]**
all models	beam	Masonry	2.04E+04	0.2	1700
	truss	Wood	1.00E+04	/	/
model 1	plate-shell	Masonry lower-band	1.40E+03	0.2	1600
	plate-shell	Masonry upper-band	6.90E+03	0.2	1600
model 2	plate-shell	Masonry lower-band	2.50E+03	0.2	1600
	plate-shell	Masonry upper-band	3.75E+03	0.2	1600
model 3	plate-shell	Masonry lower-band	4.00E+03	0.2	1600
	plate-shell	Masonry upper-band	5.00E+03	0.2	1600

Table 1 summarises the values of the mechanical characteristics used in the dynamic analysis. Models 1, 2 and 3 represent different hypotheses on the elastic modulus of the wall bands. The elastic modulus of model 1 is derived from a sonic test campaign; models 2 and 3 represent two other cases with a different ratio between the elastic modulus of the lower and upper bands.

Table 2 Modal frequencies respect the ratio E_1/E_2

	$\mathbf{E_1/E_2}$	**1st frequency [Hz]**	**2nd frequency [Hz]**	**3rd frequency [Hz]**	**4th frequency [Hz]**
model 1	0.2	1.01	1.61	2.18	2.53
model 2	0.6	1.10	1.61	2.45	2.47
model 3	0.8	1.20	1.79	2.68	2.73

In Table 2 the stiffness ratio E_1/E_2 and the values of the first four frequencies for the three models, evaluated by a standard modal analysis [1], are reported. The values of the first and the second frequency are in the range of 1.0 and 1.6 Hz, respectively, while the third and the fourth are above the value of 2.1 Hz. If the E_1/E_2 ratio ranges from 0.2 to 0.8, there is a 20% modification of the first frequency, from 1.01 Hz to 1.20 Hz, and an approximately 10% for the second, from 1.61Hz to 1.78Hz. Comparing the Young's modulus of model 1 and model 2 (see Table 1) a significant change in both E_2 and in the ratio E_1/E_2 should be noted. In this case the greater reduction (compared to the case model 1 vs. model 3) of the E_2 value, from 6.9E+03 to 3.75E+03 MPa, does not influence the second frequency (see Table 2) while it produces a change in the first frequency in the order of 10%.

Figure 3 shows the first four modal shapes associated with the eigen-frequencies presented in Table 2 in the case of model 3 (Figure 3.a-h) and of model 1 (Figure 3.i-m).

The first modal shape is a symmetrical mode with the four walls moving in phase (Figure 3.a). The sectional view (Figure 3.e) shows two different types of deformation for the central and external walls. In particular, the external walls behave similarly to a cantilever beam in the first mode while, due to the columns, the deformed shape of the central walls appears to have a concentrated rotation on the tip of the columns and an almost linear deformed shape. In Figure 3.i, the sectional view of the first modal shape of model 1 is shown to describe the influence of the E_1/E_2 ratio. Indeed, the decrease in the ratio, from 0.8 (model 3) to 0.2 (model 1), produces a deformation in the nave walls related to the bending behaviour of a plate with homogeneous material ($E_1/E_2 = 0.8$) with a cubic-type deformed shape, showing larger deformation in the lower band. This behaviour is present in all modal shapes (Figure 3.i-n).

The second modal shape (Figure 3.b) is a skew-symmetric mode, with modal nodes around the middle point of the nave; the four walls are still in phase. The

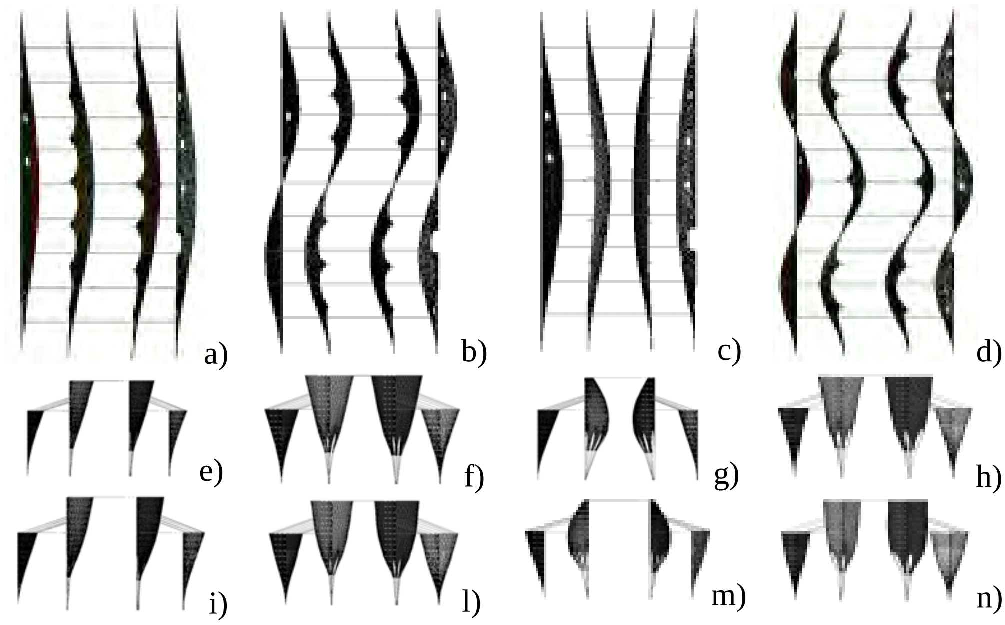


Figure 3 Modal shapes of the finite elements model: (*a*-*d*) plant view for model 3; (*e*-*h*) cross-sectional view for model 3; (*i*-*l*) cross-sectional view for model 1

small geometrical imperfections have little effect on the position of the modal node.

The third modal shape (Figure 3.c) represents a breathing-type motion with a symmetric longitudinal and a skew-symmetric transverse behaviour. In the cross-section the central and external walls have transverse displacements with opposite phases (Figure 3.g). The displacements of the tip of the columns are greater than in the first mode while the deformed shape of the upper and lower masonry bands of nave walls involves a shell behaviour more than a pure flexural-plate type behaviour characterising the first modal shape. The decrease in the E_1/E_2 ratio (Figure 3.m) with the increase of the E_1 modulus (see Table 1) makes the system more rigid: indeed less transverse displacement of the central walls are evident. It can be observed that in this case the change of the transverse displacement sign is only conventional because the modal shape still represents a breathing-type motion with alternate modal amplitude.

The fourth mode (Figure 3.d) is a longitudinal second symmetric mode, with the four walls having the transverse displacements in phase. The modal shape presents two nodes at 1/3 of the nave. Comparing the cross section representation of the modal shape of model 3 (Figure 3.h) with the one of model 1 (Figure 3.d) a linear-type deformation against a more cubic-type can still be seen.

3. DYNAMIC TEST APPARATUS

Forced vibration tests have been performed by using an instrumented hammer and a vibrodyne (Figure 4).

The characteristics of the hammer are as follows: frequency interval: 0÷500 Hz; force interval (output 5V): 0÷22000 N; average sensitivity: 0.22 mV/N; resonance frequency: 2.7 kHz; mass: 5.4 kg. Hammer blows hit walls at the level of accelerometers to give a horizontal pulse nominally perpendicular to the wall; in each test several blows were used at each point.

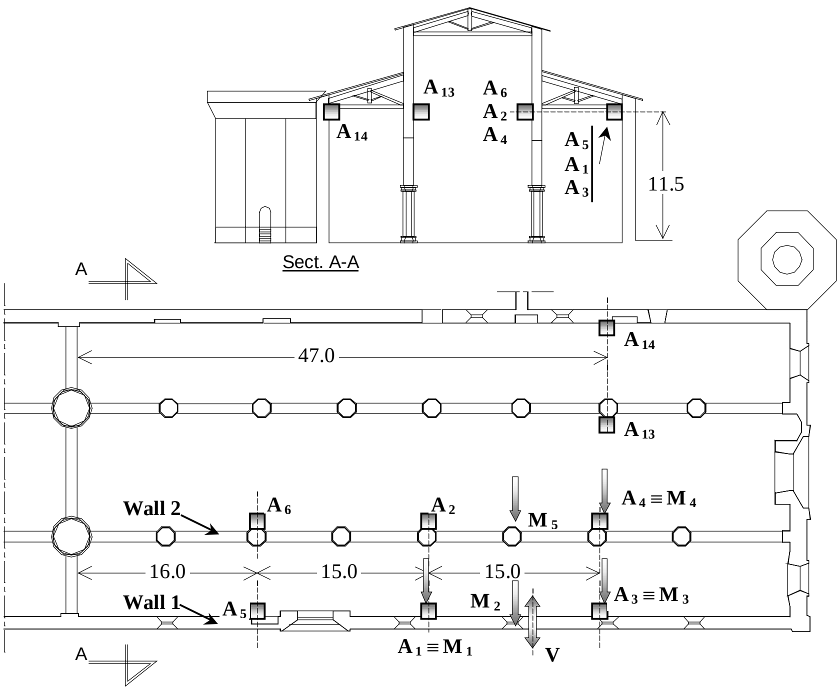


Figure 4 Dynamic test apparatus: A = accelerometers; M = hammer impact; V = vibrodyne

The vibrodyne is capable of providing sinusoidal forces within the frequency range of 0.3÷50 Hz, by using a step of 0.04 Hz, up to a maximum value of 20 KN. The vibrodyne was mounted on top of the wall, near the position of hammer impact point M_2; the frequency varied from 0.5 to 3 Hz to excite at least first two modes of the structure.

Six accelerometers (sensibility: 5 V/*g*; full scale: 2*g*; bandwidth: 0.5÷25 Hz; resolution: 0.002% of full scale) were used simultaneously in each test. Accelerometer A_{13} and A_{14} recorded only responses produced by vibrodyne.

4. DYNAMIC CHARACTERIZATION RESULTS

Recorded accelerograms were used to evaluate the transfer functions of instrumented points. Transfer functions of hammer tests were obtained by averaging the transforms of significant accelerograms, from 2 to 8 in each test, divided by the transform of the corresponding excitation [3]. Only accelerograms with coherence greater than 0.8 have been considered, so a ratio signal/noise of 12 dB is ensured. Transfer functions of vibrodyne tests were determined directly by the ratio acceleration/force at each frequency in the range investigated. The vibrodyne tests allow the transfer functions to be described more accurately. Furthermore they need less numerical handling of experimental results, but they require more complex organisation to install and to manage the exciter.

The amplitude of some typical transfer functions from hammer tests is shown in Figure 5. Even if the simultaneous examination of the phases, or the complex representation, can give a comprehensive description of the responses, the diagrams still permit the recognition of at least four major resonance peaks in the range 0.8÷3.0 Hz.

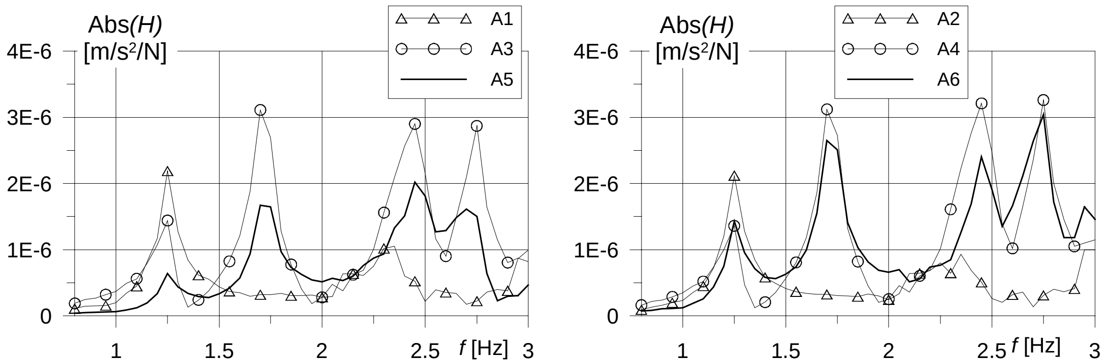


Figure 5 Experimental transfer functions, hammer test M_3

The first two peaks are around frequencies values, about 1.25 and 1.7 Hz, which are in good agreement with those computed by model 3 (Table 2). Other peaks are present over 2 Hz. Two of them, around 2.5 and 2.7 Hz, are well defined in all tests. Secondary peaks, around 2.2, 2.3 and 2.6 Hz, are not always visible in all the responses; they indicate the occurrence of highly coupled modes, that make the reconstruction of modal model over 2 Hz very complicated. These peaks, however, are estimated to be less important: numerical analysis indicates that the participating mass of first two modes is at least 85% of total mass in transverse direction of the church, when an earthquake occurs.

Due to the low level of excitations, a linear elastic behaviour can be assumed; furthermore the viscous damping is supposed to be proportional to the mass and stiffness matrix. So, the measured response data were used to develop a mathematical model of the structure defined by frequency response function $H_{ij}(f)$

evaluated by the ratio between $\ddot{u}_i(f)$, transform of recorded accelerogram at station i, and P_j (f), transform of excitation at point j:

$$H_{ij}(f) = \frac{\ddot{u}_i(f)}{P_j(f)} = \sum_s \frac{f^2 \Phi_{is} \Phi_{js}}{(f^2 - f_s^2) - 2i\zeta_s f\, f_s} \tag{1}$$

Φ_{is}, is the eigenvector element for station i, mode s; f_s and ζ_s are the s-natural frequency and damping factor respectively. The model is completely defined when the modal parameters $\chi_{ij}, = \Phi_{is}\,\Phi_{js}$, f_s and ζ_s are known. All modal parameters were evaluated as explained in [2], where one of the most effective methods for identifying modal parameters [4] has been modified to separate very close frequencies and heavy interfering modes.

The identification procedure permits a very reliable identification of frequencies and damping factors, except those of the third mode; their values determined by different hammer tests are very close to average values of Table 3.

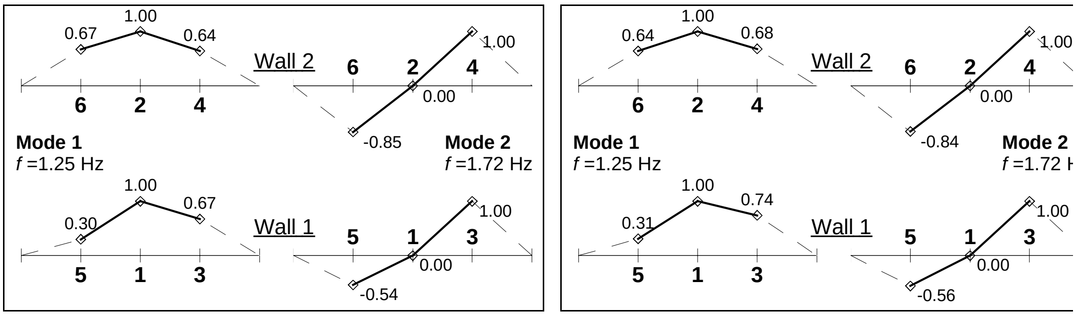
Table 3 Identified frequencies and damping factors by hammer tests

mode #	**frequency [Hz]**	**ζ [%]**
1	1.25	2.9
2	1.72	2.5
3	2.35 (?)	?
4	2.44	2.9
5	2.72	1.8

The f.r.f. identified by experimental data of Figure 5 are plotted in Figure 6, where only main frequencies below 3 Hz are considered. The diagrams show very interesting information that can be summarised as follows.

If only the first two peaks are considered, it is clear that there is an excellent match between the responses of accelerometers A1 and A2, as well as A3 and A4, which are in pairs in the same transverse alignment on external and internal walls. This behaviour demonstrates the efficiency of the wooden trusses that link walls. On the contrary, the response of A5 is considerably lower than A6, thus manifesting a relative displacement caused by a lack of link in the roof restraints, even at low excitations.

Even if their contributions to the whole response of structure are less important, the good agreement between pairs A1-A2 and A3-A4 can be observed also over 2 Hz, whereas the different response in the pair A5-A6 is confirmed.



a) b)

Figure 7 Identified modal shapes by M3 (a) and vibrodyne (b) tests

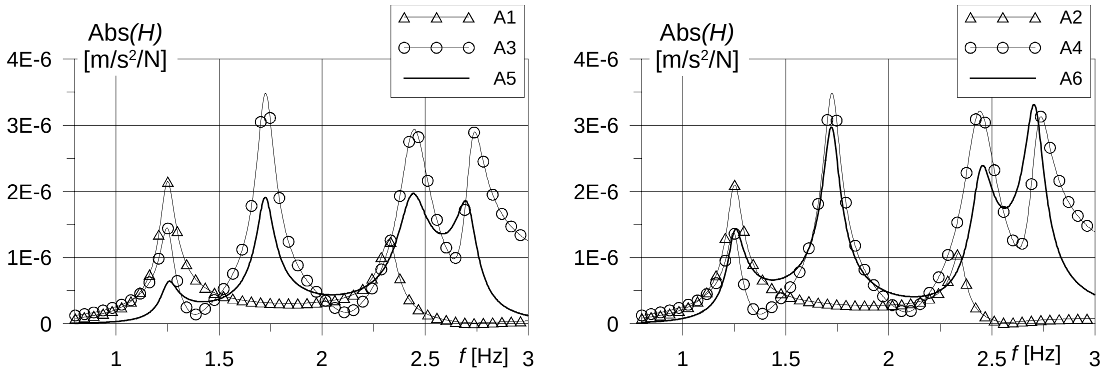


Figure 6 Identified transfer functions, hammer test M_3

The first two frequencies measured by vibrodyne tests were found to be f_1 = 1.22 Hz and f_2 = 1.67 Hz on average; damping factors were found to be in the range of 1÷2%.

Modal shapes, identified by hammer and the vibrodyne tests, are compared in Figure 7; displacements at the edges of walls are assumed null, due to the stiffness of facade and transept. The agreement is excellent.

In both cases there is a difference between identified displacements in the pair A5-A6, already observed in Figure 6, is evident. This behaviour can be attributed to the presence of a large door in the external wall, positioned between accelerometer A5 and transept. The door creates a localised area with reduced stiffness that limits the energy transmitted to transept zone whose rigidity alters the response of the adjacent wall.

The good agreement of modal parameters evaluated from hammer and vibrodyne tests demonstrates the efficiency of hammer excitation: it can be considered a less expensive and less complicated alternative to vibrodyne, which should only be used when an accurate excitation is required.

5. CONCLUSIONS

The dynamic behaviour of the Basilica S. Maria di Collemaggio was analysed using both numerical and experimental methods.

The contemporary use of FE and experimental modal analysis allowed a more accurate numerical prediction to be made and pointed out some deficiencies in the connections of the roof to the walls, notwithstanding the low level of excitations. Even if the linear numerical models allowed the framing of mainly features of the behaviour, it seems very sensitive to the values of mechanical parameters; only dynamic tests showed the actual behaviour of structure where nonlinearity is present even at low excitations and the links, constraints and elements can not be always correctly modelled.

The good agreement between hammer and vibrodyne tests demonstrated the possibility of using a less expensive and less complicated hammer when an accurate knowledge of excitation is not required.

The repetition of dynamic tests in the future and the comparison with present analysis will allow the evaluation of the small scale repairs work in progress and, more generally, to monitor the efficiency of the Basilica

ACKNOWLEDGEMENTS

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THE SOIL STIFFNESS INFLUENCE AND THE EARTHQUAKE EFFECTS ON THE COLOSSEUM IN ROMA

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ABSTRACT

Structural analyses, carried out on mathematical models based on the original “flavian” configuration and taking into account different possible soil stratification and nature, highlight the influence of the soil characteristics on the stress distribution over the whole structure. Further analyses have shown how the first earthquakes (III century) had induced so high stress to justify the first damages and the subsequent main collapses in the Southeast sector of the structure.

1. THE FIRST CENTURIES

The Flavious Amphitheatre, built on the bottom of the Labicana Valley, was in a first time inaugurated in the year 80 a.C. during Emperor Tito’s kingdom. The foundations stand on a heterogeneous ground not completely known (and now under study) formed by recent Holocene alluvial deposits less compact and less resisting then the underneath Pliocene stratification.
In the first period, about 140 years, several adverse events have involved the monument, nowadays known as Colosseum, and had considerable effects on the structural stability. The reduced soil stiffness, not homogeneously distributed under the monument, is the cause of some first differential settlements, relative movements and the consequent increasing of local stresses. Thus during the first earthquakes the concentration of the major damages took place on the southern part of the monument.
In the following centuries the not symmetric development of the damages and collapses is evident in the present shape of the monument. According to historic sources, great damages took place with the fire in the year 217;

important restoration works were carried out from 218 to 223 and were completed only in 238 (stopping the games in the Severian time). But probably these damages were amplified by the earthquakes in the year 217 (Dione Cassio, but uncertain) and 223 (9 September or 19 October): the *Chronicon Paschale* defines these seismic events as very strong (we can evaluate them in the range of the 5th grade of the Mercalli Scale), while in the *Historia Augusta*, *Al. Sev. 44.8*, it is mentioned that Alessandro Severo was particularly economically engaged in repairs after such earthquakes.
This situation is due to the previously reached not symmetrical weakness of the structure and to the many following more intense earthquakes, to the omission of maintenance increased since the VI century and to the habit to remove parts of the monument (for safety measures in case of localised instability or for reuse in other structures).

2.THE MATHEMATICAL ANALYSES OF THE STRUCTURAL BEHAVIOUR

The study of the structural behaviour of the monument refers to the original geometry of the Flavian age and takes into account the interaction with the low stiffness of the soil and the first earthquake effects.
For the structural analyses at this stage of the study, elastic global models, employing solid finite elements (figure 1), have been considered.

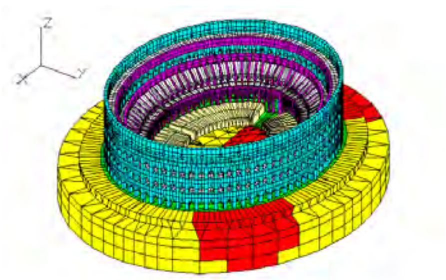


Figure 1

FLAVIAN AMPHITHEATRE AND FOUNDATIONAL SOIL MODEL

These models represent, besides the structure and the foundations (figure 2) of the monuments, also the soil until the bedrock (figure 3).

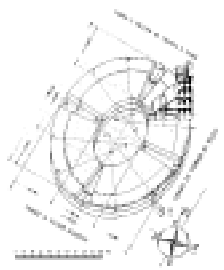
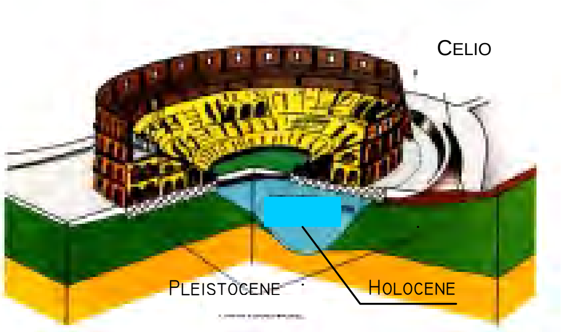


Figure 2

THE FOUNDATIONS OF MONUMENTS

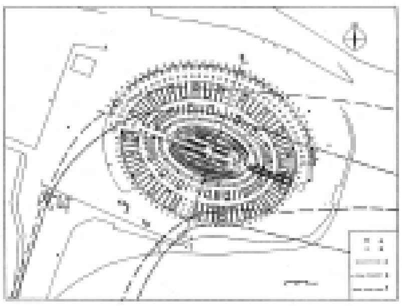


THE SCHEMATIC GEOMORPHOLOGY OF THE SOIL ADOPTED FOR THE "FLAVIAN" MODEL

Figure 3

The stratigraphic soil situation, not yet well known for lack of specific geotechnical inspections and tests, has been represented according to the hypothesis of the most recent studies on the Labicana valley [5 and 9]. These hypotheses are based on general regional evaluations as well as on stratigraphical data of neighbouring areas and on the results of three deep inspections in the Arena foundational soil [12].
Under the southern base area of the Colosseum there are less stiff soil strata in a curvilinear trend as the Labicano ancient stream that has originated those strata. Conversely on the northern side the foundations rest on more resistant and stiff sediments (figure 4, see also figure 3). To these two different foundational soils correspond, in the models, different stiffness for which are tried different reciprocal ratio (i.e. 1:1, 1:5 up to 1:16) in such a way to take into account the present uncertainty of knowledge.

THE SCHEMATIC GEOMORPHOLOGY OF THE SOIL ADOPTED FOR THE "FLAVIAN" MODEL



——— *Remarkable orographic profiles on the surface*

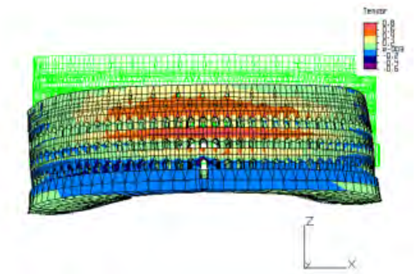
- - - - *Limits of the alluvial Holocene valley*

– – – -*Limits of ancient alluvial "middle" Pleistocene valley*

Figure 4

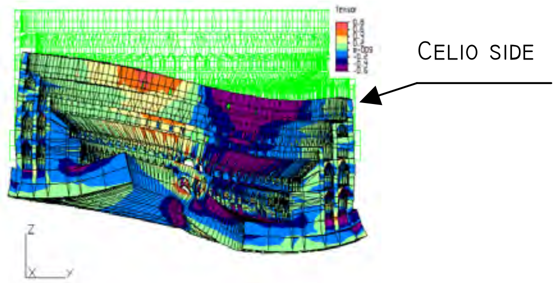
As regard the definition of the seismic actions, considered in all the models, it has been taken into account the historical data relative to the registered events (7^{th} – 9^{th} grade M.S. with a return period of about 300 – 500 years) as well as the studies of the seismic characteristics of the roman seismological region. Such studies agree in the definition for a return period of 400 – 500 years, the

maximum “magnitude” equal to about 7th grade M.S. and the highest acceleration equal to 0.05 – 0.06g.. To such values correspond a response spectrum of 0.15 - 0.16g, for structures with their own period of 0.4 seconds.
The results of the analyses relative to the original configuration (the Flavian time structure) clearly show that the damages, with the distribution and succession of collapses historically documented, are due both to the non homogeneous stiffness of the soil (figure 5 and figure 6)



ANNULAR TENSILE STRESSES AND DEFORMED SHAPE OF THE OUTER WALL (NORTH SIGHT)

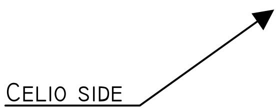
Figure 5



ANNULAR TENSILE STRESSES AND DEFORMED SHAPE (EAST SIDE FROM INSIDE)

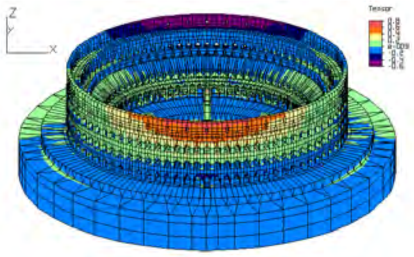
Figure 6

and to seismic actions producing adverse combined effects in the area of the east entrance and in the southern sector (fig. 7 and 8).



σ_Y STRESSES IN CASE OF EARTHQUAKE IN THE MAIN AXIS DIRECTION

Figure 7



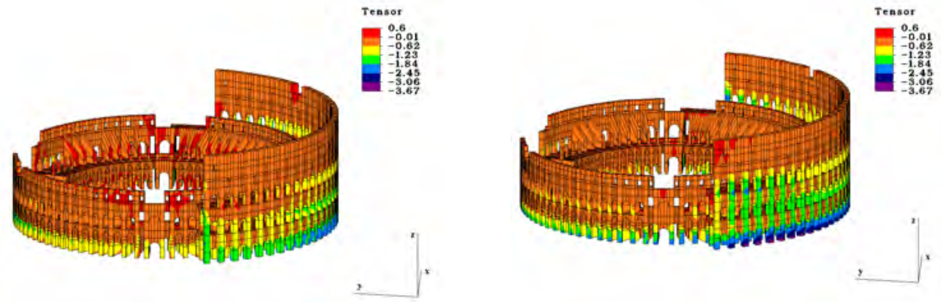
σ_x STRESSES IN CASE OF EARTHQUAKE
IN THE MINOR AXIS DIRECTION

Figure 8

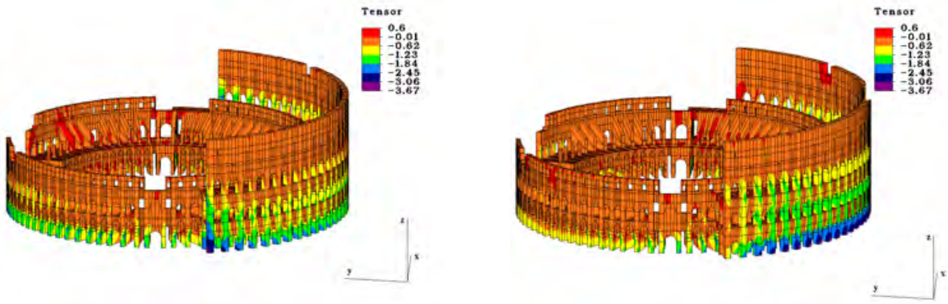
3.CONCLUSIONS

The settlements and the soil movements would not have been able to produce, alone, the registered collapses, but certainly they contributed to make more critical the effects of the first earthquakes. The following, along the centuries, successive earthquakes (fig. 9), together with the progressive lack of maintenance and the subsequent violations, have caused the development of asymmetrical damages and collapses, mainly spread on the Celio side according to the historical documentation and the present situation (fig. 10, 11 and 12).

MODEL OF THE STRUCTURES AT THE END OF THE XVIII CENTURY: VERTICAL STRESSES



EARTHQUAKE IN THE MAIN AXIS DIRECTION



EARTHQUAKE IN THE MINOR AXIS DIRECTION

Figure 9

Model of the structures in the present condition: vertical stresses

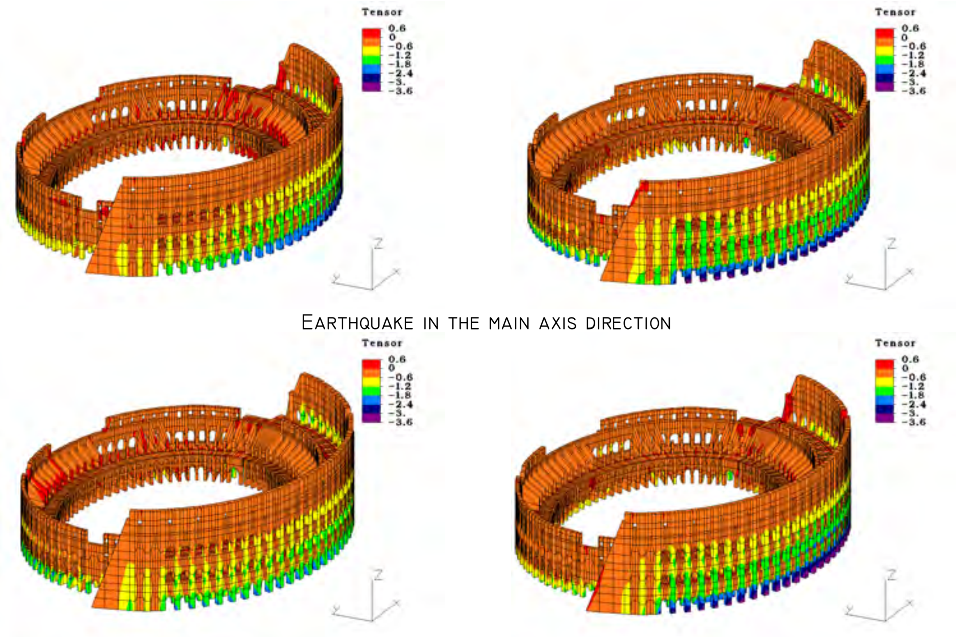
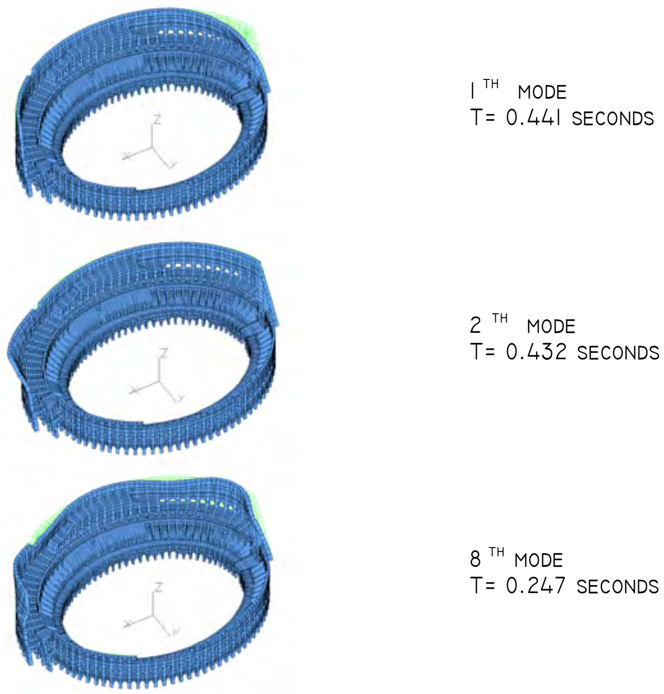


Figure 10

Model of the structures in the present conditions



MODAL ANALYSIS

Figure 11

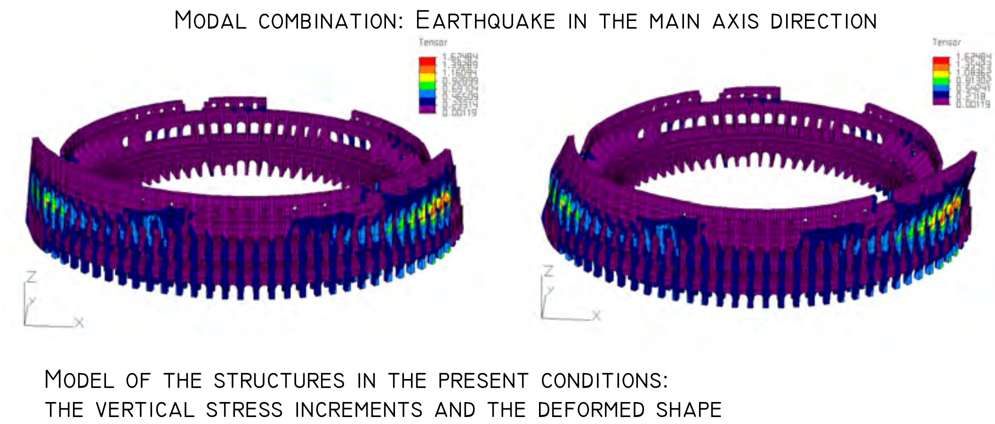


Figure 12

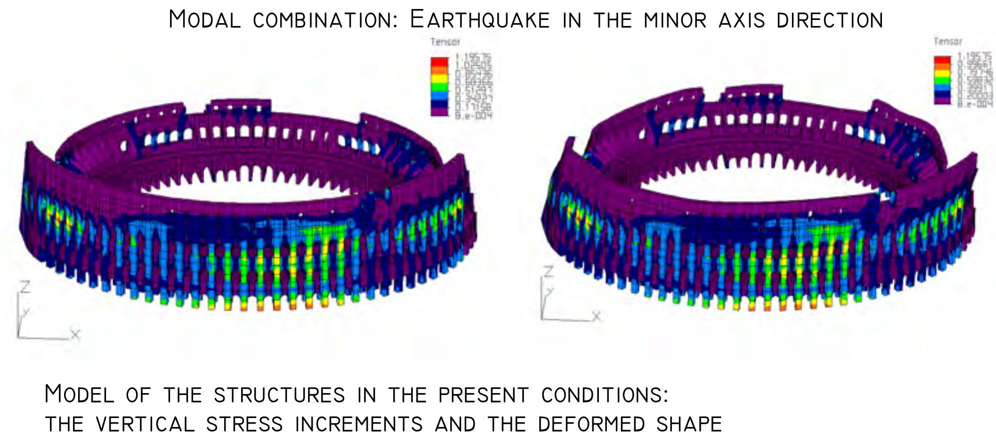


Figure 13

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ANALYSES OF SEISMIC RELIABILITY OF THE MASONRY CONSTRUCTIONS BUILT ON THE DIOCLETIAN'S PALACE IN SPLIT (CROATIA)

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ABSTRACT

An automatic procedure of relief usually carried out to evaluate the seismic vulnerability of masonry buildings has been improved and applied to structural aggregates built up, in many ages, in the Palace of Roman Emperor Diocletian, in Split (Croatia), between the forth century a.C. and the nineteenth century, which represent a extraordinary historical testimony of the town.

1. INTRODUCTION

In the procedures of analysis of the seismic vulnerability of the masonry buildings, according to the methodologies running employed in Italy, the relief operations on one side constitute the more onerous operating phase from the point of view of the necessary time and the economic costs, from the other the more difficult and delicate phase from the point of view of the reliability of the successive evaluation of vulnerability. Handbook and predisposed technical forms for such relieves constitute profits guides, but, in particular if used from detectors not particularly expert, can carry to wrong or not reliable classifications in the rigid outlines of the prefixed categories from the analysis methodologies. Categories that often do not concur to hold account of the enormous variety of the masonry walls typologies, floors and roofs realised in different zones and/or different periods, and of the same ambiguity of the terminology employed for their description.

In the within of the construction of an expert system for the analysis of seismic vulnerability of buildings in masonry, they have been put to point computer science procedures that concur the interactive access of the detector with bases of acquaintance and bases of data, orienting the observation of building the more important characteristics of the typology in examination and automatically

recording the result in the format demanded for the further analyses of vulnerability.
The relief procedure and the corresponding analysis of vulnerability, in this phase, has interested a champion of 63 masonry buildings, pertaining to zones south-east of the Diocletian's Palace, in the town of Split (Croatia) (Fig. 1-6).
Several in-situ tests have been carried out on different masonry walls, with different typologies (from the Diocletian's walls of the 4-th century, to 8-th century ones, to medieval ones, until 19-th century ones (Fig. 7)) in order to calibrate the use of the expert system and to became possible the evaluations of structural reliability and the forecast of the effects of different hypotheses of retrofitting.

2. VULNERABILITY ANALYSES

The procedure, created in Padova University [8-15] is based on a vulnerability model of masonry buildings, that depend on the following parameters:
I1 is the ratio of in-plane shear strength of the walls system to total weight;
I2 is the ratio of out-of-plane flexural strength of the most critical external wall to total weight, evaluated by summing the resistance of vertical (I2') and horizontal (I2'') strips;
I3 is the weighted sum of the scores of seven partial vulnerability factors;
a is the mean absolute acceleration response of the building;
u is the uncertainty factor depending through a fuzzy relation on I3.

The output vulnerability Vu = f (I1, I2, a, u) is the probability of collapse or damage ≥ degree D4 (EMS98: European Macro-seismic Scale 1998).
The evaluation of the vulnerability of each building requires the definition of rules to merge qualitative and quantitative informations. Two aspects must be solved: the relation between I3, synthesising qualitative judgements on the characteristics of the building and the parameter u (of which the reliability function f_S = 1-Vu depends); the influence of the quality of informations on the final measure of f_S.
The fuzzy set theory [30, 32] has been adopted for such purposes: it gives general formal rules to treat vague informations which have been already applied in the field of structural reliability. Details of the procedure are given in [8] and the results can be given, for each value of the seismic intensity a, and for each building, through the membership function of the fuzzy subset of interval of variation of f_S.
The building can be assigned to fixed classes of vulnerability using numerical techniques for ordered classification of the fuzzy subset fs. In the application five classes of vulnerability (Very Small, Small, Medium, Large, Very Large) to which the fuzzy subsets are associated, are considered.
The results can be used not only to compare the vulnerability of every single building of a group (Vu) but also the vulnerability of different groups of buildings

(Vg); in the second case the statistical distribution of the buildings of a sample in the classes can be used. Both the types of comparison are useful when retrofitting interventions are to be planned in a region, given for every group of buildings the expected intensity of the seismic action.
Shaking table tests on masonry buildings models [3] show that in the highly damaged state the seismic intensity a is nearly equal to PGA.

To the current state the complex is introduced like a altogether apparently homogenous take-over for constructive typology. In the truth, in phase of relief, above all from the examination of the inner surfaces of masonry, they have been found, in a great number of cases, situations of walls with various masonry types much in the same wall (Fig. 8), with insertions of far ages much between they, from the Roman Diocletian's wall, of IV the century, to the clay bricks wall of VIII the century to the medieval, until renaissance, baroque and finally twentieth-century examples. The prevailing use of the squared or roughly hewn stones, arranged in regular way in all the facades, can draw in deceit on the real distribution of the mechanical characteristics of the carrying elements. The presence of a second poorer quality and more worse organised inner layer, let alone the frequent presence of a internal zone of material with insufficient consistency, even if modest thickness, alters the theoretical carrying capacity considerably, is for forces in the plane of the wall (compression for vertical loads, shear for horizontal actions) that for those out of plane.
To completion of the general information on the characteristics of the built up complex, it goes added that the buildings are developed with articulated aggregations a lot in plant and often also in elevation and denounce various states of conservation, prevailing one be of generally poor maintenance. The expert system has concurred variable times of relief from one - two hours for simple buildings to three - four hours for buildings with articulated plant.
The masonries are realised mostly in quasi-regular fabric squared or roughly hewn stones of good quality (Fig. 9-10), of limestone origin; the original mortar of aerial lime is rather porous and has medium resistance to the superficial abrasion. The state of conservation of the same one is extremely variable, like often happens in the historical centres, depending mostly on the state of exposure and maintenance of the masonry surface. In the zones of insufficient maintenance and with high humidity degree the mortar turns out nearly always of much poor quality.
In attended of further experimental results, in the treatment of the data they have been assumed temporarily, for the materials, values that for affinity with previous surveys of the search unit, correspond to three different levels of mechanical characteristics of masonry (in the order: compression strengths = 2.00, 3.00, 4.00 MPa; tensile strengths = 0.10, 0.15, 0.20 MPa; density = 2300, 2500, 1800 kg/m^3).
The analysis of seismic vulnerability of the single buildings, conduct with fuzzy approach is carried out for increasing values of seismic intensity, measured from

the medium value of the response acceleration of the building, between 0.08g and 0.40g. Tale field comprises, for a reason or purpose pure indicative, also values 0.16 g, 0.28 g and 0.40 g, suggested from the Italian Code for the verification of masonry buildings of new construction respectively in zones of third, second and first category. The vulnerability is measured through a qualitative judgement to 5 levels: Very Small, Small, Medium, Large and Very Large. In Fig.11 are brought back the classes of vulnerability of the single examined buildings, for increasing values of answer acceleration (0.08 g, 0.12 g, 0.16 g, 0.28 g, 0.40 g), and for a medium value of the "class of uncertainty ", for every building. In a detailed report of next publication the analyses of vulnerability of the single buildings related with the data of II level GNDT forms will be introduced also.

3. CONCLUSIONS

The judgements on the seismic vulnerability of each building, also being born from analysis of collapse mechanisms connected to horizontal actions, can naturally be correlated with the state of static efficiency, and in last analysis with the reliability of the same construction also under current conditions. In other words, the poor resistance to the horizontal actions, that it is involved the operation of the resistant walls is for shear that (above all) for bending - due to a poor control of the horizontal structures (possibility of slip of the wooden rafters of floors), to the lack (also partial) of perimeter r.c. beams or iron chains - reveals a inborn weakness of the whole construction. One elevated vulnerability class can indicate therefore also, plus generally, one greater dangerousness of the state of the construction. The vulnerability maps can therefore be one qualitative instrument for the appraisal of the "state" of an aggregate of buildings.
At the current state of the research, in the vulnerability analyses have been used values of the mechanical characteristics of masonry deduced for analogy from previous experiences, mediated through the interpretation of the tests carried out in situ. In extending and detailing relationship, currently in preparation, analyses of different carried out vulnerabilities placing to comparison will be exposed also hypothesis on the materials, considering also the cases in which interventions of consolidation of masonry and of buildings in their complex are operated (insertion of chains, formation of perimeter beams, etc), let alone the forecasts of damage for every building, for several scenarios of seismic intensity. In both these analyses, lead with the aid of the expert system previously cited, will distinguish also the "relative classes of uncertainty " to every building, described like fuzzy sets through the information deduced from the compilation of II level GNDT forms.
Returning to the description of the activities of the unit of research of Padova University, it goes remembered that the survey team has carried out between April and May 2000 the collection of the necessary information (carrying relief of geometry, elements, floors, materials, cracks, degree of conservation, etc.) on a

total of 63 buildings (23 in zone 5 and 40 in zone 10) in correspondence of the wall south of the Diocletian's Palace.
In attended of one more deepened and careful appraisal of result of tests carried out in some buildings, that it will constitute the base for the analyses of vulnerabilities included in a detailed report that the unit of research is preparing, remain temporarily acceptable the choices operated previously (and prudently) for analogy with previous experiences. Otherwise, the adopted fuzzy approach for the expert system is particularly adapted to the treatment of "uncertain" informations, as they are those on the mechanical characteristics of the materials and/or on the resistances of the structural elements.
It is intention of the group of research, in accordance with the municipality of Split, to extend the relief and the analyses of seismic vulnerability also to the buildings of the other zones contained in the perimeter of the Diocletian's Palace.

Acknowledgements

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Fig.1 - Aerial view of Diocletian’s Palace in Split

Fig. 2 - View of a part of the south-west wall of Diocletian’s Palace

Fig.3 - View of east wall of Diocletian’s Palace

Fig.4 - View of south quarter of Diocletian’s Palace

Fig.5 - View of south-west quarter of Diocletian’s Palace

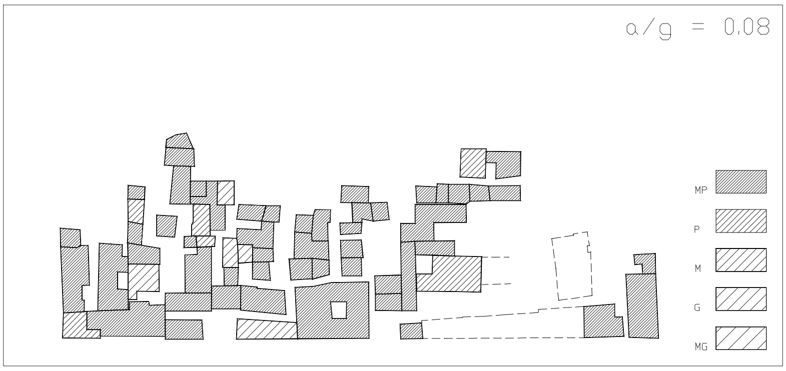
Fig.6 - View of two masonry buildings of south-east corner of Diocletian’s Palace

Fig.7 - Construction periods of masonry buildings of Historic Core of Split

Fig.8 - Wall with inserts of Diocletian’s masonry

Fig.9 -


Fig.10 -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P
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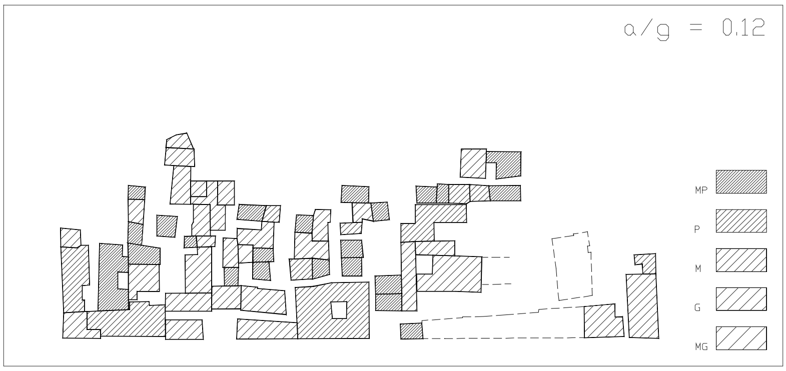
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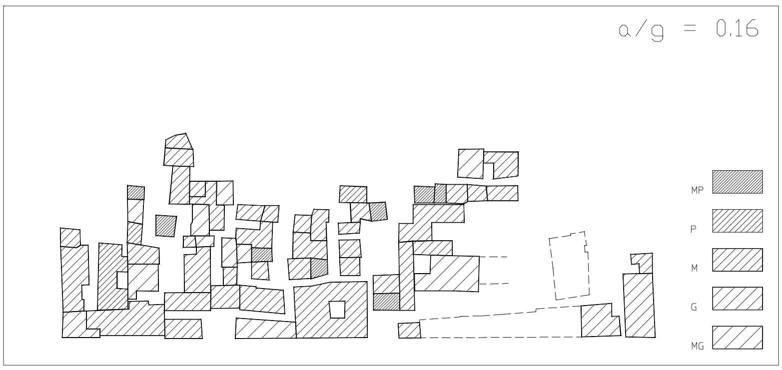
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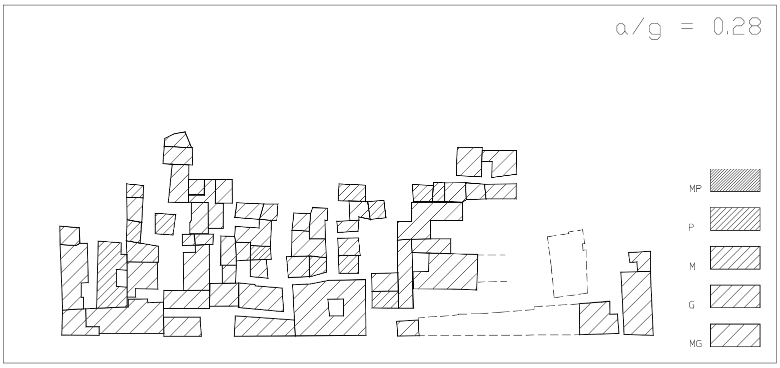

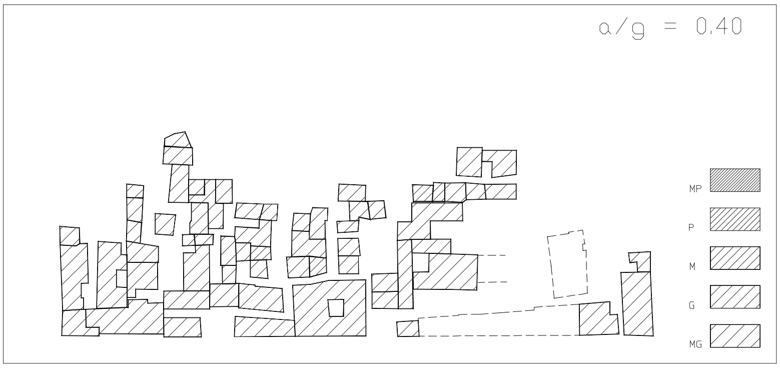

Fig. 11 – Vulnerability classes for south Diocletian’s Palace masonry buildings (Five classes, from Very Small (MP) to Very Large (MG)) [25].

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THE RESPONSE OF MODELS OF ANCIENT COLUMNS AND COLONNADES UNDER HORIZONTAL FORCES WITH OR WITHOUT SMAD's

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ABSTRACT

Results and conclusions are presented from an experimental and numerical study that examines the response of simple models of ancient columns or colonnades. The influence on the response arising from the inclusion of wires having energy dissipation characteristics (SMA wires) is also studied. The excessive rocking and sliding and subsequent collapse of the epistyle is an additional form of unstable response in addition to the excessive rocking, rotation and sliding of the individual columns. The insertion of the SMA wires seems to inhibit, up to a point, unstable modes of response, whereas these identical model structures without the wires developed certain types of unstable response at lower excitation amplitudes. The numerical simulation used to predict the pull-out test response of the colonnade structural formation with or without SMA wires seems to reproduce the most significant aspects of the observed response.

1. INTRODUCTION - STUDIED STRUCTURAL CONFIGURATIONS

Ancient Greek peripheral temples composed of large heavy members that simply lie on top of each other in a perfect-fit construction without the use of connecting mortar, are distinctly different from relatively flexible contemporary structures. The dynamic and earthquake behavior of this type of structural systems is simulated in the present study by utilizing specimens that are relatively rigid and are developing deformations mainly by rocking and sliding response at their supporting boundaries. The employed rigid bodies for forming these models were made of steel and are assumed to be models of prototype structures 20 times larger. Three basic configurations are examined here, as outlined in the following:

1.1. Model Single Steel Column.

The first studied structural configuration is that of a model single steel truncate cone assumed to be a model of a monolithic free-standing column, as shown in figures 1 and 2. As reported elsewhere by Manos and Demosthenous (1991, 1997), these model columns, as all steel columns which were employed in the formation of model colonnades that are described next, were manufactured in such a way that they could represent prototype columns either sliced in drums or monolithic columns. For the tests reported here, for the single steel column or for the steel column colonnades, all model columns are monolithic.

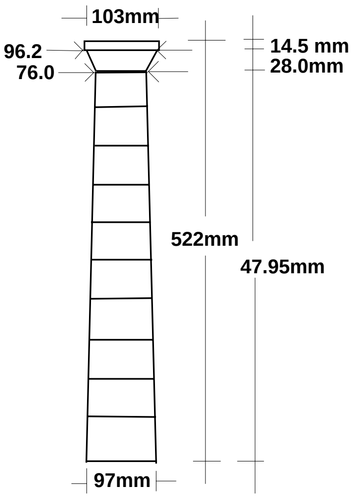

Figure 1. Geometry of the single steel column

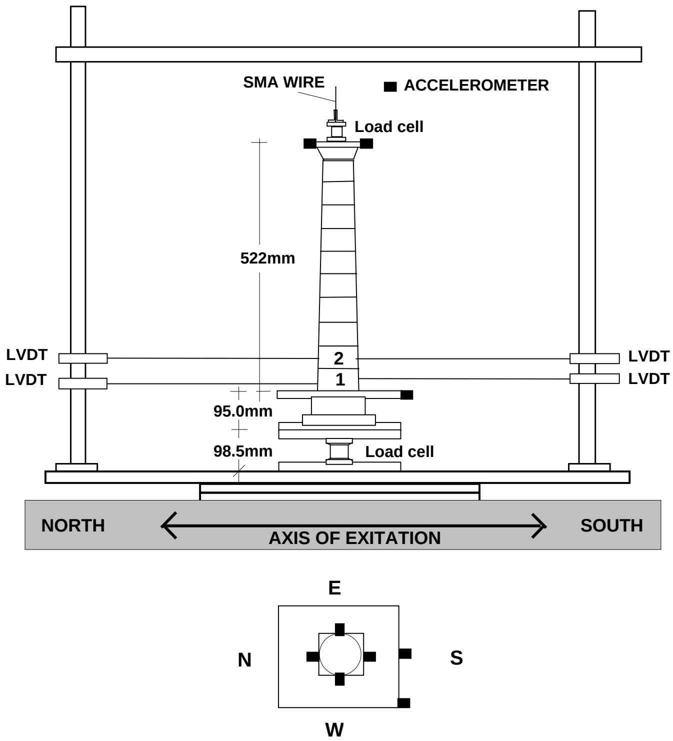

Figure 2 Single steel column on the shaking table

1.2. Two-Steel Column Model Colonnade.

The second studied structural configuration is formed by two steel truncate cones, of the same geometry as the model of the individual column described in 1.1 before, but supporting a rectangle of solid steel at the top, representing in this way the simplest unit of a colonnade; this is shown in figure 3. In this second configuration the weight of the epistyle was varied. This variation of the weight of the epistyle results in numerous sub-formations from which two distinct cases are reported here; one with this weight of the epistyle being equal to 286Nt whereas the second had the weight of 605Nt. The centers of this two-steel column model colonnade coincided with the axis of the horizontal base motion.

1.3. Four-Steel Column Model Colonnade.

The third configuration is again a model colonnade formed by four steel truncate cones with identical geometry to the ones used before. A monolithic rectangular steel epistyle with dimensions shown in figures 4a and 4b was placed at the top of these identical four steel columns in a perfect fit condition. This model structural formation is depicted in figure 4a showing the four column assembly looked at from the top of the epistyle as it is resting on the shaking table moving platform. Indicated in figure 4a is the relevant orientation of this model formation with the North (N) - South (S) direction coinciding with the direction of the horizontal motion of the shaking table. As can be seen in figures 4a and 4b the four-steel column model colonnade is symmetric with respect to this N-S axis of horizontal excitation. It can also be considered as a twin of the two-steel column model colonnade described in 1.2 before, because of the identical geometry and the weight of the epistyle on the four-column formation which is approximately twice as much of the weight of the epistyle for the two-column formation. Some details on the geometry and the instrumentation that was employed in this testing arrangement with the four-column formation is also depicted in figures 4a and 4b.

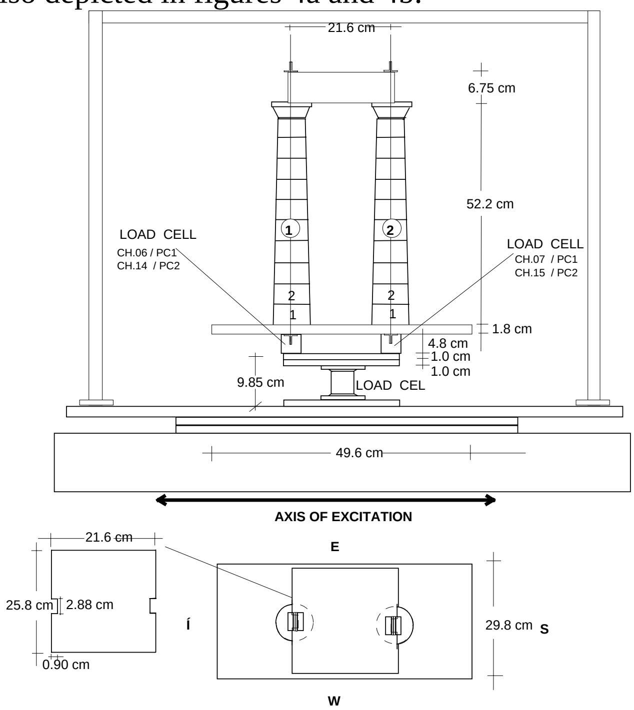

Figure 3. Two-steel column model colonnade

1.4. Configurations of Columns and Colonnades with Wires

The experimental investigation was supplemented by an additional study that aimed to examine influences on the dynamic response arising from a certain intervention technique with devices based on Shape Memory Alloy (SMA) wires having energy dissipation characteristics. These devices were applied at critical locations and aimed to provide an increased resistance as well as energy dissipation capacity (figures 2,3,4a and 4b). The wires that are used throughout were 1mm in diameter and were provided by FIP Industriale of Padova, Italy, in the framework of a cooperative research project supported by the European Union (Manos, 1998). Their mechanical properties were fully investigated by a special testing campaign conducted at the Joint Research Center of the European Union at Ispra, Italy. A limited number of basic tests were also conducted at the Laboratory of Strength of Materials of Aristotle University with the wires that were directly employed in the present investigation.

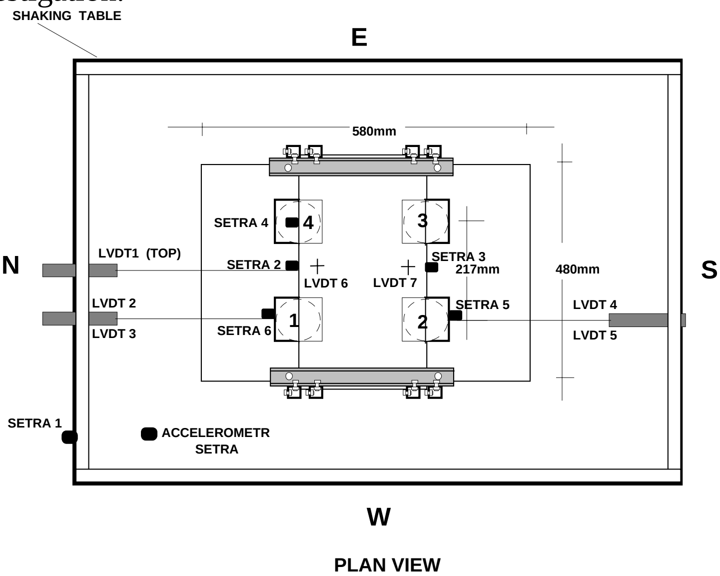

Figure 4a. Four-column colonnade (Plan View)

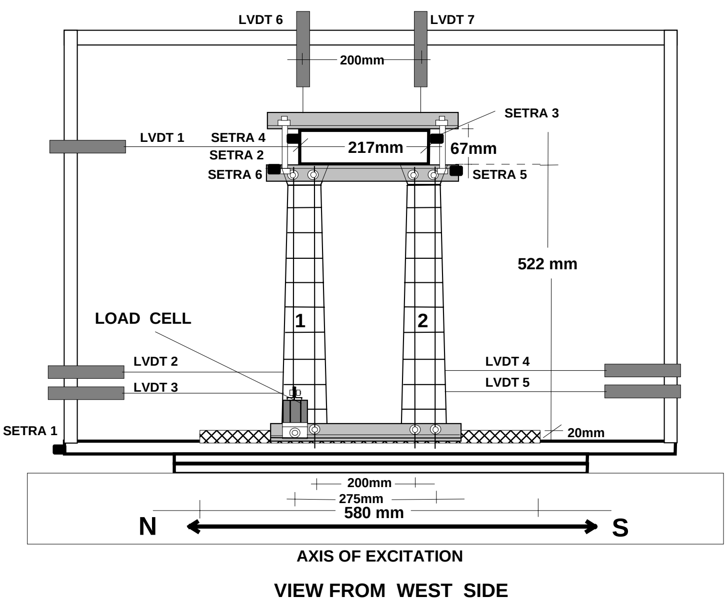

Figure 4b Four column colonnade. Testing arrangement along the axis of the horizontal base motion.

2. EXPERIMENTAL INVESTIGATION

During this experimental sequence the models described in 1.1, 1.2 and 1.3 are subjected to a variety of base motions before and after the intervention technique with the SMA wires. During testing, acceleration and displacement measurements were recorded in order to identify sliding and rocking modes of response. A very stiff, light metal frame was built around the studied model structure in order to carry the displacement transducers that measured the rocking angle; this metal frame also provided temporary support to the specimen during excessive rocking displacements indicating overturning. The sequence of tests included a series of sinusoidal base excitations as well as earthquake simulated tests. Moreover, through a series of relatively strong intensity base motions, the stability of the model formations was studied together with the resulting collapse modes at certain stages, focusing on the influence that the inclusion of the SMA wires has on such collapse modes. Finally, the dynamic and simulated earthquake base motions were supplemented with tests named 'Pull Out Static Tests'. During these tests a well controlled horizontal displacement was imposed at the top of the studied model formation at a slow rate. The displacement response of the model was monitored together with the horizontal load that resulted at the top from the gradually imposed horizontal displacement.

3. OBTAINED EXPERIMENTAL MEASUREMENTS

3.1. Model single steel column without SMA wires.

3.1.1. Sinusoidal Tests : During these tests the frequency of motion was varied from 1.5Hz to 4Hz. This resulted in groups of tests with constant frequency for the horizontal sinusoidal motion for each test. In the various tests belonging to the same group of constant frequency, the amplitude of the excitation was varied progressively from test to test. Summary maximum response results from such tests are depicted in plots such as these of figure 5a. The following points can be made from the observed behavior during these tests:

- For small amplitude tests the rocking behavior is not present; the motion of the specimen in this case follows that of the base.
- As the horizontal base motion is increased in amplitude, rocking is initiated. This rocking appears to be sub-harmonic in the initial stages and becomes harmonic at the later stages.
- Further increase in the amplitude of the base motion results in excessive rocking response, which, after certain buildup, leads to the overturning of the specimen. At this stage the rocking response is also accompanied by some significant sliding at the base as well as by rotation and rocking response out-of-plane of the excitation axis.

Summary results for the single steel column without any wires are depicted in the plot of figure 5a. The ordinates in this plot represent the amplitude of the base acceleration whereas the absiscae represent the frequency of the base horizontal dynamic excitation. The following observations summarize the main points as they can be deduced from this plot:

- The stable-unstable limit rocking amplitude increases rapidly with the excitation frequency.

- For small values of the excitation frequency the transition stage from no-rocking to overturning, in terms of amplitude, is very small and it occurs with minor amplitude increases.

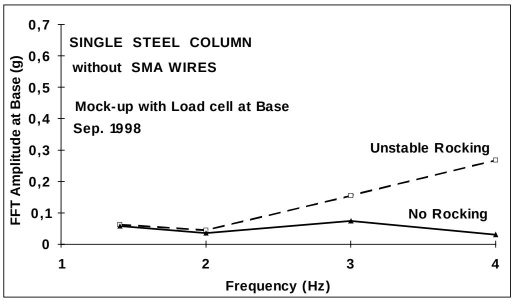

Figure 5a Single column dynamic response without SMA wires

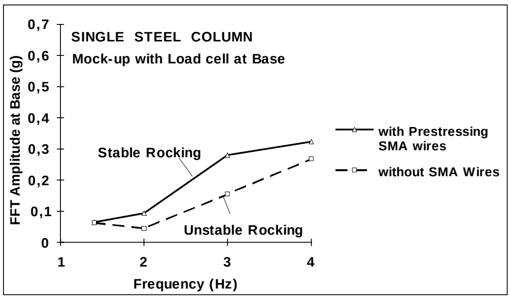

Figure 5b Single column dynamic response with SMA wires

3.1.2. Simulated Earthquake tests : A number of tests were performed during this sequence with progressively increasing intensity. The principal objective of these tests was to observe again the stable-unstable behavior of the model which exhibited similar trends to the ones discussed in 3.1.1.

3.2. Model single steel column with SMA wires.

3.2.1. Sinusoidal and Simulated Earthquake Tests. Tests similar to the ones described above were performed for the single column with one SMA wire passing through the center of the column (figure 2). Due to space limitations only summary results are presented here from the testing sequence with the sinusoidal horizontal base motions. The response curve obtained from these tests for the single steel column with the SMA wire is depicted in figure 5b together with the corresponding response curve of the single steel column without the SMA wire. As already mentioned, when discussing the response curve of the single steel column without the SMA wire of figure 5a, this curve represents in this case a boundary between stable-unstable rocking response. When this boundary is exceeded, because of an increase of the amplitude of the base motion of constant frequency, it leads to the overturning of the model structure (instability). In contrast, the plotted response curve for the mode structure with the SMA wire does not represent a stable-unstable boundary. Because of certain limitations in the capacity of the shaking table and in order to protect the SMA wires and their anchoring fixtures from repeated damage from overturning, the stable-unstable boundary was not established in this case. Instead, the plotted curve indicates amplitudes of the base motion with the model structure fitted with the SMA wire still exhibiting stable rocking response. Obviously, the stable-unstable boundary for the model structure with the SMA wire is expected to occur at higher amplitudes of the base excitation than those represented by the plotted curve, which in this case is designated as stable rocking response. By comparing the stable rocking response curve in figure 5b for the model single steel column with the SMA wire with the stable-unstable boundary for the same structure without the SMA wire (figure 5a), the favourable influence of the insertion of the SMA wire on the stability of the dynamic sinusoidal response can be clearly identified. Similar favourable influence of the insertion of the SMA wire on the stability of the model structure was observed during the earthquake simulated tests.

3.2.2. 'Pull Out Static Tests' This testing arrangement was described in paragraph 2 before. The obtained response in terms of non-dimensional rocking angle (ordinates) and applied horizontal load at the top of the single steel column with the SMA wire (absiscae) is depicted in figure 6. As can be seen from figure 6 the insertion of the SMA wire leads to an almost elastoplastic force-displacement response; moreover the unloading path is accompanied by a lower plateau and a recovery of the plastic strain similar to the one observed during the uniaxial tests of the individual SMA wire.

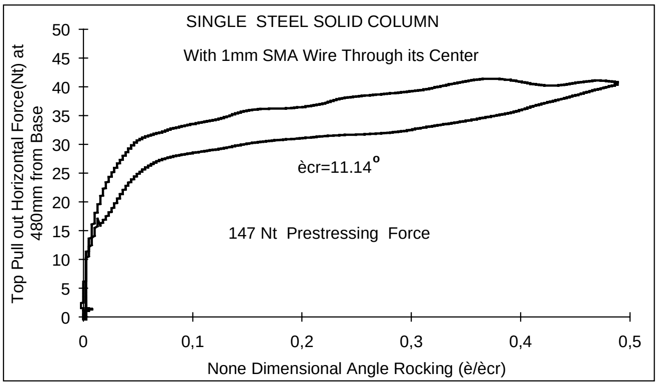

Figure 6. Static pull-out response of single steel column with SMA wire

3.3. Model colonnade with two or four-steel columns and with or without SMA wires.

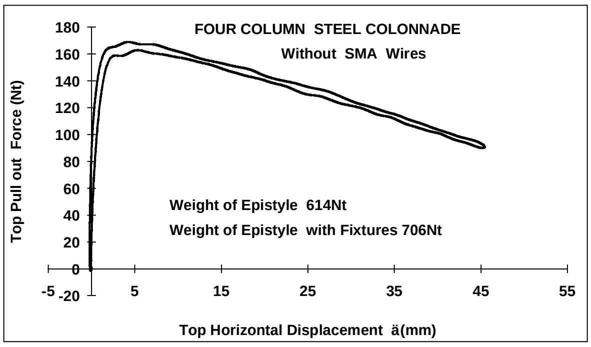

Figure 7a. Static pull-out response of four-steel column without SMA wires

3.3.1. Sinusoidal base motions and Simulated Earthquake Tests. The testing sequence based on the sinusoidal and simulated earthquake excitations which was described in 3.1.1. and 3.2.1. before is also repeated here. As can be seen from the results obtained so far the performance of the model colonnades, exhibit similar stability trends with that observed for the individual column. The excessive rocking and sliding and subsequent collapse of the epistyle is an additional form of unstable response in addition to the excessive rocking, rotation and sliding of the individual columns.

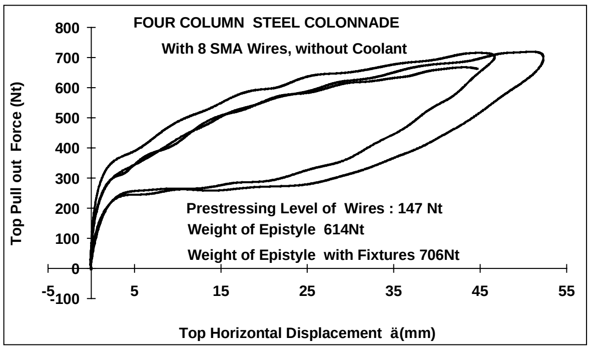

Figure 7b. Static pull-out response of
four-steel column with 8 SMA wires

<u>3.3.2. Pull-out static tests.</u> As mentioned in 3.2.2. before, this testing arrangement (described also in paragraph 2) was utilized in subjecting colonnades of various configurations to static pull-out horizontal force, which was applied at the center of the epistyle. Figure 7a depicts the measured response for a four-steel-column formation without any SMA wires whereas figure 7b shows the corresponding measured response of the same formation when eight (8) SMA wires were added. By comparing the measured behavior of the four-column colonnade with and without the inclusion of the SMA wires in these two figures the following points can be made.

- The addition of the SMA wires increased the capacity in terms of horizontal force.
- The formation without the SMA wires exhibits a descending branch after the maximum horizontal load is reached. In contrast, the formation with the SMA wires demonstrates that a post-yielding type of behavior is initiated for displacements exceeding the initial elastic-type behavior. This post-yielding type of behavior is accompanied by continuously increasing horizontal load.
- Finally, the loading - unloading cyclic measured behavior of the four-column colonnade with the SMA wires, demonstrates clearly its dissipative nature, which is of course due to the fact that, for this level of deformation, the corresponding mechanical characteristics of the used SMA wires are mobilized.

4. NUMERICAL SIMULATION OF THE STATIC PULL-OUT RESPONSE

In this paragraph are presented some of the results obtained by an extensive numerical study which aims to predict the behavior of the single steel column and the various colonnade formations when they are subjected to horizontal static pull-out force, as described in sections 3.2.2. and 3.3.2., respectively. The interface between the columns and the top block as well as the columns and the ground support was approximated by frictional contact-elements, which are part of a specific software package. These elements could only sustain compression together with friction forces whereas no tensile forces could develop. For the configurations including SMA wires the former contact elements were combined with elasto-plastic springs approximating the measured mechanical properties of the SMA wires. The columns and the epistyle were approximated with 2-D quadrilateral plane-stress elements. Figure 8 shows the mesh employed for the two-column colonnade. The latter can also be used to approximate the four-column colonnade, which, due to symmetry, can be considered as a twin two-column colonnade (see figures 4a and 4b, section 1.3). The numerical analyses were performed by applying the desired level of imposed displacement to the same location that was used in the experimental sequence.

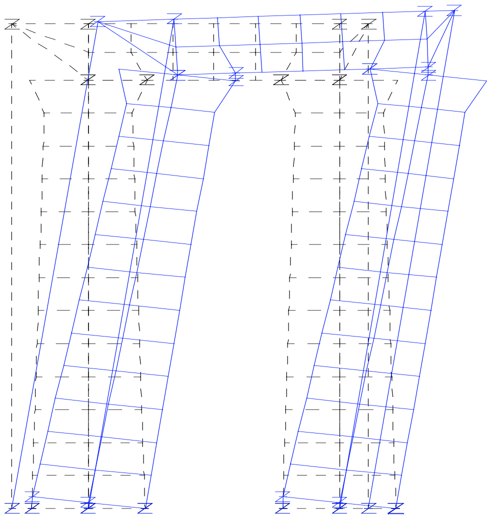

Figure 8. FEM mesh for the two steel and four-steel column colonnade

Due to the geometric and material non-linearities of the contact elements and the elasto-plastic springs the solution, nonlinear in nature, followed many steps whereby the level of horizontal deformation was increased gradually in each step, with a considerable number of iterations per step. In this way, a simple attempt was made to numerically simulate both the contact problem that is inherent in the behavior of the studied structural formations as well as the presence of the SMA wires, which were installed in the way described for these structural formations. The objective here was to be able to check if such a relatively simple numerical simulation could yield realistic results so that then it can be utilized in the framework of our research effort. Parametric studies were also performed in order to check the sensitivity of the solution to various parameters. However, space limitations do not allow its inclusion here. The numerical results that are presented here belong to the numerical simulation of the behavior of the four-steel column colonnade with and without SMA wires. The horizontal load - horizontal deformation predicted response is depicted below together with the corresponding measurements in figure 9a and 9b.

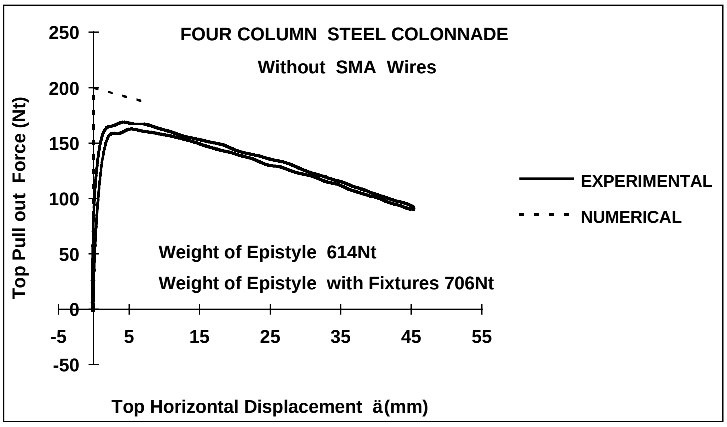

Figure 9a. Measured and predicted static pull-out response of four-steel column colonnade without SMA wires

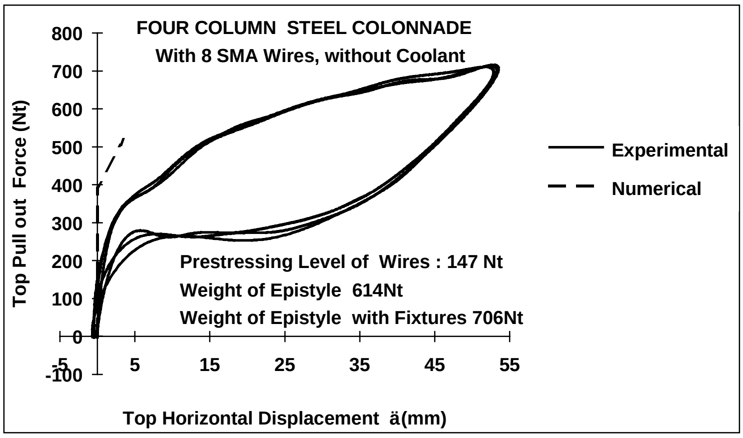

Figure 9b. Measured and predicted static pull-out response of four-steel column colonnade with eight (8) SMA wires

- The predicted response of the four-column formation without the SMA wires appears to reproduce in a satisfactory manner both the peak horizontal load as well as the descending branch of the response.

- Given the simplicity of the numerical approximation for the four-column formation with the SMA wires, the obtained numerical response reproduces in a realistic manner qualitatively if not quantitatively the most significant aspects of the response with the SMA wires in terms of maximum horizontal load values and ascending load path. However, no attempt has yet been made in order to predict numerically the observed cyclic dissipative behavior.

5. CONCLUSIONS

- The insertion of the SMA wires as described had a noticeable favourable influence on the stability of the studied model formations. The model structures with the insertion of the SMA wires developed stable response at amplitudes higher than those at which the model structures without SMA wires had already overturned. This is more apparent for relatively lower frequencies.

- The dissipative behavior of the structural formations with the inclusion of the SMA wires was clearly demonstrated with the static pull-out tests.

- The numerical predictions of the four-steel column colonnade with or without SMA wires reproduces in a realistic manner the most significant aspects of the observed response.

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EXPERIMENT AND ANALYSIS ON THE ASEISMATIC BEHAVIOR OF XI'AN BELL TOWER

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KEYWORDS

Ancient Architecture, Bell Tower, Wooden Structure, Dynamic Characteristic, Aseismatic Behavior, Historical City

ABSTRACT

Xi'an is a historical and cultural city. From eleventh century B.C. to 907 A.D, Xi'an and Xi'an area served as a capital for thirteen dynasties, spanning over 1080 years. Many buildings, including palaces, temples, tombs and mausoleums were built on or under the ground. Most of them beneath the ground are well preserved up to now, but unfortunately, many buildings on the ground, including the mausoleums were destroyed. In this paper, the natural damages of some buildings are described briefly, then proceed from the structural features of Xi'an Bell Tower, the aseismatic mechanism of the ancient wooden structures of China is approached and a vibration model of the structure is established. Based on the modern aseismatic theory, and by means of the computer analysis and dynamic experimental technique, the dynamic characteristics, the aseismatic capacity and the aseismatic behavior of a Bell Tower situated in the center of Xi'an city are given. At last, introduce some buildings, which have been reconstructed in Xi'an of China. The conclusions could be regarded as a referring basis for protecting ancient architecture in China.

1. INTRODUCTION

Xi'an was the capitals of thirteen dynasties in ancient China, about one hundred kings and emperors had been lived here. It had developed economy and culture in the ancient. But numerous buildings were destroyed in wars. Nowadays some large buildings were built in Tang, Ming, and Qing dynasties. They last for three hundreds to a thousand years. Owing to Xi'an is a seismic area, it is very valuable that these buildings could be seismic area, it is very valuable that these buildings could be preserved. So it is an important work to study and protect these buildings using aseismatic theory and modern technology.

The ancient buildings in Xi'an can be divided into two kinds. One is the brick masonry structure such as big wild goose pagoda, small wild goose pagoda erected in Tang dynasty in 652 A.D. and 707 A.D. and Xi'an city wall built in Ming dynasty in 1374~1378 A.D. The small wild goose pagoda having a height of 45 meters cracked into two halves in the Shaan'xi earthquake in 1487, but did not collapse. The city wall has been repaired.

The another one is the wooden building, such as Bell Tower erected in Ming dynasty in 1384 and rebuilt in 1582, Drum Tower built in 1384, four city gate tower and the great mosque. By way of example, we study a typical wooden structure of Ming dynasty, the Bell Tower.

2. THE EVOLUTION AND STRUCTURAL FEATURES OF BELL TOWER

Bell Tower is 36 meters high and its architectural area is 1378 square meters, standing in the center of Xi'an city as the symbol of this city. It was originally built in 1384, the beginning of Ming dynasty, dating back to 600 years ago. In 1582 it was rebuilt 26 years after the major earthquake of Shaan'xi in 1556. Since 1582, four times earthquakes over magnitude 4 (on the Richter scale) had happened in Xi'an area, and in those earthquakes, the aseismatic capacity and the aseismatic behavior of Bell Tower structure were excellent. But it is uncertain whether this tower was destroyed or not in those disasters, and whether it can also withstand considerable earthquake or not in the future.

According to the historical record engraved on stone tablet, the rebuilt Bell Tower is almost the same as the original one except the base of the building made higher. So the structures of the original and the rebuilt one are the same. Fig.1 is its current photograph.Fig.2 is the photographic surveying figure of this tower.

In addition, a bird sight photo of Bell Tower and its environment are taken from 7000 meters high above it as a reference for the planning of Bell Tower environment [1].

Bell Tower's plane configuration is square. The pier foundation, the principal part of structure and the tile roof named Zan-Ding constitute it. The pier foundation is 8 meters and 5 decimeters in height and is built with blue bricks. The side length of the pier foundation is 35 meters and 5 decimeters. There are two vertical crossing arch culverts of 6 me

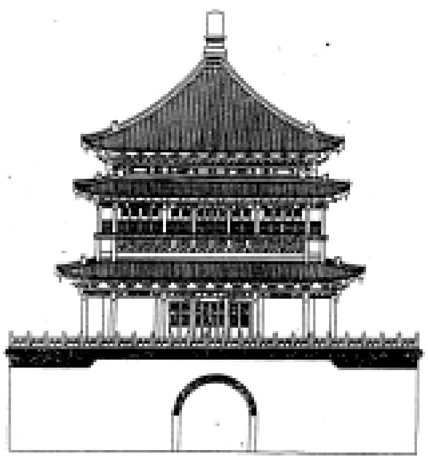

Fig.1 Bell Tower in the center of Xi’an Fig.2 the photographic surveying of Bell Tower

outside. The column footings are put on the pier foundation by means of the square stone templates, and the column caps are joggled respectively with corresponding wooden beams of the roof. Between the columns, there are several cross beams of fair depth-span ratio, and the columns are joggled with these cross beams to form two space frames, which seems as a tube in tube. Fig.3 is the plane and sectional drawing of Bell Tower [2].

The mechanical properties of Bell Tower's structure are very excellent. For instance, Its center of mass and center of geometrical stiffness are located at the same point, so the effect of torsion is very little; Wooden structures are relatively light in weight, so their seismic forces at each story are corresponding small; The application of the pendentive joints situated eaves not only decorates the building, but also gradually makes above loads transmit to the columns, to reduce the local stress of the joints and improve the ductility of the structure; The reasonable slip of the column footings have a good effect on shock absorption; The mortise-and-tenon joints connecting the columns and the cross beams make the Bell Tower become a compact and relatively flexible structure. When a earthquake occurs, by way of the sliding friction and extruding deformation of mortise-and-tenon and pendentive joints as well as the reasonable slip of the column footings, the seismic energy can be considerably worn down and absorbed, therefore a serious damage of the structure can be prevented and the structure won't collapse under violent earthquake.

3. THE VIBRATION MODE OF ANCIENT WOODEN STRUCTURE [3]

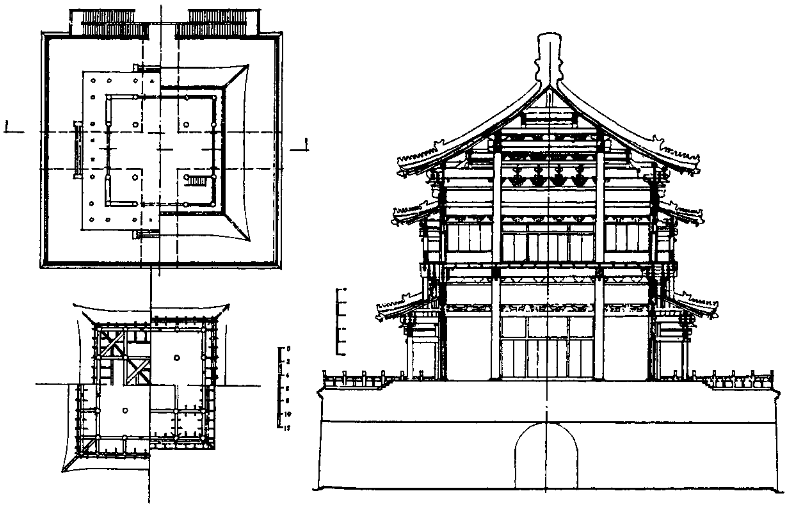

Fig.3 the plane and sectional drawing of Bell Tower

The column footings of Chinese ancient wooden structure are generally put on the brick or soil pier foundation by means of the stone templates with or without groove, so they can suitably slip as well as turn under the action of seismic force. Based on this, the vibration mode of ancient wooden structures is shown in Fig.4. When damping is ignored, the dynamic equations of the system can be written as follows:

$$k(x - x_1 - h\varphi) = m\omega^2 x \qquad (1)$$
$$k_1 x_1 = m\omega^2 x \qquad (2)$$
$$k_2\varphi = m\omega^2 xh \qquad (3)$$

Substituting Eqs. (2) and (3) into (1), the following equation is given

$$m\omega^2\left(\frac{1}{k} + \frac{1}{k_1} + \frac{h^2}{k_2}\right) = 1 \quad (4)$$

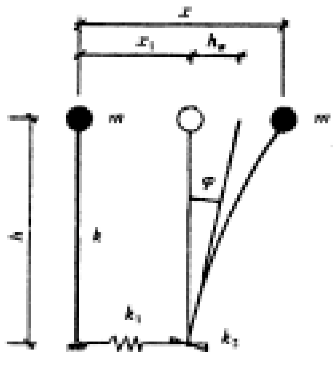

Fig.4
the vibration mode of ancient wooden structure

Assume $$k^* = \frac{1}{\frac{1}{k} + \frac{1}{k_1} + \frac{h^2}{k_2}} \qquad (5)$$

Then $$m\omega^2 = k^*, \omega^2 = \frac{k^*}{m} \qquad (6)$$

Where k^*=composite stiffness of the system, namely series stiffness of k, k_1, and k_2.

Case 1. When k_2 is relative large, the column footings can't turn but elastically slip, so $k^* = kh_1/(k + k_1)$;

Case 2. When k_1 is relative large, the column footings can't slip but elastically turn, so $k^* = k\ k_2/(k_2 + kh^2)$;

Case 3. When k_1 and k_2 are relative large, the column footings can neither turn nor slip,i.e., the general elastic single-particle Vibration system, therefore $k^*=k$;

Case 4. When k_1<<k or k_2<<k, $k^* = k_1k_2/(k_1 + k_2)$.

Based on the analyses above, it is shown that the elastic slip and turn of the column footings can make the 1st natural frequency of the system cut down by a big margin.

4. THE ASEISMATIC CAPACITY OF BELL TOWER

1). The basic hypotheses:

(1). Under horizontal force, the ends of the columns and the beams can bear a certain moment, that is to say, they can be regarded as rigid connection in calculation;

(2). Under vertical load, the ends of the beams can't bear any moment but the ends of the columns, i.e., the ends of the columns and the beams can be respectively regarded as rigid connection and hinged connection.

(3). The connections between the column footings and the pier foundation can be regarded hinged connection, i.e., k_1 and k_2 can be respectively regarded as very large and very small.

2). The calculating sketch

Choose a typical plane frame, and ignore the effects of the less important members, then the calculating sketch of Bell Tower' structure under the action of horizontal force is shown in Fig.5. Table 1 shows the actual geometrical parameters of the columns and the beams concerned, in which, the wood is northeast oak of China, then $\rho = 766\,kg/m^3$, $E = 15.50\times10^3\ N/mm^2$.

3). The calculating results

Based on the calculating model as noted above, using the response-spectrum method of multi-degree-of -freedom system to calculate, the bottom shear of Bell Tower's structure under eight-degree earthquake and classification soil is 309.2kN. This value is far small compared with the bottom anti-slide capacity of the 16 columns. In addition, on the basis of the above mentioned

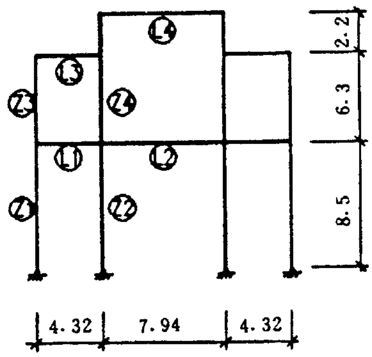

Fig.5 the calculating sketch of Bell Tower calculating model and the current design standard concerned [4], the internal forces and the bearing capacity of the columns and the beams concerned under vertical load is computed. The calculating results are shown in Table 1.

Table 1 the actual geometrical parameters and the static calculating results of Bell Tower's structure

members	Z_1	Z_2	Z_3	Z_4	L_1	L_2	L_3	L_4
section/mm	d=500	d=700	d=500	d=700	d=300×700	d=300 ×800	d=300×700	d=300×800
axial forces/kN	261.7	300.6						
moments/kN·m					108.2	366.9	122.0	286.8
R/S	11.6	19.77	>11.6	>19.77	16.64	9.60	1.25	1.99

The results of calculation show that the seismic response of the structure is relative small, and the aseismatic capacity and the static bearing capacity of Bell Tower's structure are satisfied with the requirements of the current design standard concerned.

5. THE DYNAMIC CHARACTERISTICS AND THE ASEISMATIC BEHAVIOR OF BELL TOWER

The Bell Tower is a Complex wooden structure. In order to study this structure further, we wrote a special finite element programme on dynamic analysis for this kind of space structure. And a organic glass scale model of its main structure with the ratio of 1:30 was made. It is showed in Fig.6. By means of a French Prdera multi-point exciting experimental system and using the analogical ratio between the scale model with its prototype, the Bell Tower's dynamic characteristics were obtained and aseismatic behavior was analysed further. Fig.6 shows the model in dynamic experiment.

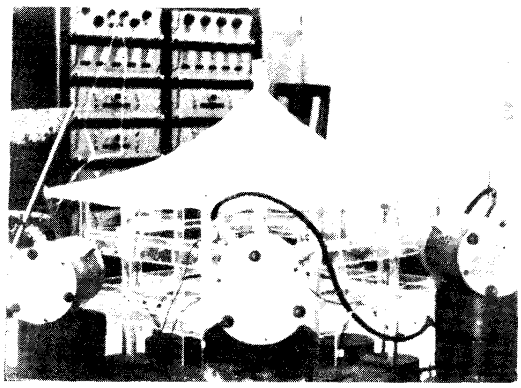

Fig.6 the multi-point exciting experiment

The results of prototype calculation and model dynamic experiment show that this tower has a quite great aseismatic capacity and a good deformed performance, and it won't seriously damage or collapse under violent earthquake. But it still has some disadvantages. The stiffness of Bell Tower in diagonal and torsion is weak. Fig.7 shows the 1st and 2nd

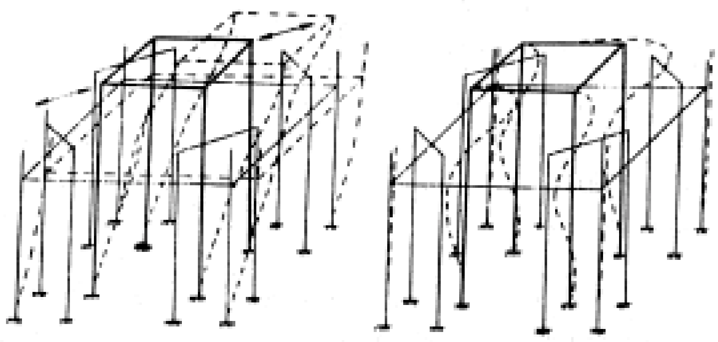

Fig.7 the 1st and 2nd modes of diagonal bending vibration modes of diagonal bending vibration. Fig.8 shows the 1st and 2nd modes of torsion vibration. Fig.9 are the modes of rhombic vibration obtained from calculation and model experimentation. In the experiment, there are relatively great diagonal bending vibration. This should be specially strengthened in Bell Tower's aseismatic reinforcement.

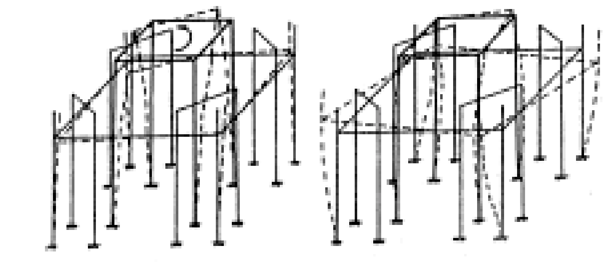

Fig.8 the 1st and 2nd modes of torsion vibration

The same as traditional architecture in north China, Bell Tower has a big and heavy tile roof. On one hand, it provide a heavy mass towards the top of the building, so when violent earthquake happens, a considerable inertial force comes to being at the top of the building, and this inertial force will make the structure have a large stress or displacement, even collapse. The Bell Tower's roof is 15kN per meter square, and its plane surface is 22×22 meter square, so its total weight is about 7200kN. With such a big mass on top of a structure, it is certainly not a good fact. On the other hand, the heavy mass of the tile roof will make the 1st natural period of bending vibration and the damping larger, and the big roof with better stiffness will make the entire structure have a good aseismatic behavior. Before, the heavy roofs of some ancient buildings were replaced with some light roofs in repairing reinforcement, but in violent earthquake, the damages of these buildings were very serious, and most of them had collapsed. We should keep firmly in mind about these bitter lessons. The ancient wooden structure has it's own constructional characteristics. Before bearing pressure, the truss of structure has quite variable nature in geometry; only after bearing a certain pressure, the joints between the members will tend to become compact, thus the truss will have an ability to withstand horizontal force and displacement. Moreover, the mass of the heavy roof is also the essential condition to keep the column footings reasonably slipping. In fact, owing to the particularity of dynamic characteristic of the ancient wooden structures, the influence of the heavy roof on seismic response is not very obvious, and this response generally does not play a part in control to the reliability of the structure. With regard to the function of the big and heavy roof is worth studying further.

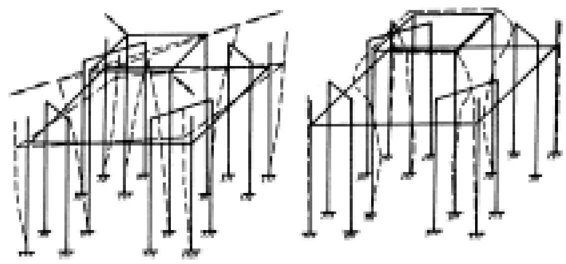

Fig.9 the 1st and 2nd modes of rhombic vibration

In addition, the ancient wooden structure has itself defects. For example, the mortise-and-tenon joints will cut down the cross section of the columns and beams at both ends, and the members may be pulled out or torn provided be unreasonably installed; The wood is easily rotten not giving a good protection or treatment. These must be paid attention enough.

Other wooden ancient structures in Xi'an are the four city gate towers named East, West, North and South. They were built in Ming dynasty and repaired recently. They are the important parts of ancient Xi'an city wall.

6. CONCLUSIONS

In Chinese history, Shaan'xi Province was the ancient political center, and Xi'an is an ancient historical cultural capital. Before 11th century, there had been developed economy, civilization and a large amount of great architecture here, and at present Shaan'xi, especially Xi'an is well known for its innumerable historical heritages and ancient monuments. According to the archaeological investigation, there are over 3000 ancient ruins and buildings, which remain to be research in Shaan'xi. The ancient architecture in Shaan'xi holds a special position in the Chinese architectural history. Although most of them had collapsed, there are also some of the ancient buildings remained. Some of them remained after many major earthquakes. So it is an important work to study and protect these structures by means of modern aseismatic theory and technology.

In this paper, the Bell Tower — the symbol of Xi'an is studied as a typical ancient building. A finite element programme on dynamic analysis is written and a scale model of this tower is made. By ways of calculation, analysis and experiment, we know that: mortise-and-tenon joints, pendentive joints and column footing joints of Chinese ancient building have a good effect on shock absorption, they are the main factors to keep building not collapsing in violent earthquake; Big and heavy tile roof of Chinese ancient building has a important effect on improving roof stiffness and entire stability of structure, so it isn't reasonable to be replaced with light one; The aseismatic capacity and the static bearing capacity of Bell Tower's structure are satisfied with the requirements of the current design standard concerned. In the aspect of mechanical analysis, the necessity for further studying ancient architecture is emphasized. These results can also be applied to other ancient buildings' study of structural characteristics and aseismatic behavior.

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DYNAMIC, STATIC AND STABILITY ANALYSES OF A MINARET STRUCTURE

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ABSTRACT

In this paper linear elastic dynamic, static and stability analyses and design of the stone masonry structure of the medieval Tombul mosque minaret in Schumen town are presented. Its lower part is cracked. A refined cantilever plane beam-column structural model, the FEM and a special computer program are used. The seismic load, the dynamic and resonance wind loads are determinate by the response spectrum method. The higher-order bending moments on the deflection beam axis and the safety factor against static buckling are obtained by the iterative Vianello's method. The bearing capacity is checked. The effect of many factors is estimated. The theoretical results are compared with available full-scale dynamic experimental results. Conclusions are done.

1. INTRODUCTION

The dynamic analysis of high-rise structures of a little supporting area such as water, TV and radio towers, industrial chimneys, minarets etc. is ordinary performed by the plane vertical cantilever beam-column model. Different mathematical methods (the Jacobi's matrix method, the Ritz's vector method and others) are initially used to obtain the frequences and mode shapes of the free undamped vibrations for linear systems of many DOF by the computer procedures. The harmonic analysis is appropriate to study the forced undamped harmonic or periodic vibrations. The mode superposition method is applied to decompose the partial differential equation set of motions in the linear steady-state analysis under complex loads and actions as well in the study of the damped free or forced harmonic vibrations. Two main approaches are then utilized for dynamic analysis. In the first approach analytical solutions of the partial differential equations of the beam-column motions with distributed and concentrated masses are get by an direct integration, the initial parameter method or the displacement method. The second one is a discrete and more often implemented approach for the structures with finite number DOF by the last two methods also or by numerical methods (first of all the FEM). The horizontal vibrations are usually taken into account.

Two fundamental approaches for seismic and wind dynamic analyses of the frame structures are mainly applied. The response spectrum method for linear seismic analysis of the structures is widely used. The time history nonlinear or linear analysis by direct integration of the structure dynamic equations in the discrete time domain using special or general step-by-step numerical methods and procedures and given accelerogramms or wind pulsation curves is applied for especially responsible and important structures situated in higher seismic and wind activity regions. In the two cases the structure can be discretized by the FEM.

The beam-column static stability can be checked applying the FEM or the displacement method for static analysis together with an iterative method or the load incremental path method and taking into account the higher-order bending moments. Special computer programs as [1] or general multipurpose program systems can be used for structural analysis and design of high-rise tower-type structures by the plane beam-column model.

In [2] an seismic analysis by the response spectrum method and redesign of an original cracked minaret structure and the strengthened structure in Eger (Hungary) is presented. The vertical plane cantilever beam-column model of concentrated masses is used.

2. DESCRIPTION AND CONDITIONS OF THE MINARET STRUCTURE

Tombul mosque has been constructed in about 1743. The mosque minaret is 38,74 m high. From outside it consists of five parts in height (Fig. 1 a-d): an underground foundation, an aboveground lower thickening rectangle prismatic base, an irregular 16-sided truncate pyramidal junction part, a main high regular 16-lateral prismatic part with a truncate 16-sided pyramidal widening (muezzin's balcony) and a conical wooden roof. The minaret internal surface is as 16-lateral prismatic. There are no data for the foundation and the soil conditions. Till the +8,65 m level the minaret is joined to the mosque lower body but this connection is structural up to the +5,57 m level only. The main high prismatic part has a circumcircle diameter of 1,70÷1,53 m and wall thickness of 23,0÷18,5 cm.

The aboveground minaret structure up to the +30,55 m level is constructed by stone masonry. The blocks are finely whittled off from hard limestone. The laboratory tests of the several removed specimens show that the stone average volume weight is of 20,5 kN/m^3; the average compressive, tensile and shear crushing strengths are as 10,3 MPa, 1,1 MPa and 2,6 MPa respectively. The mortar average strength is of 0,2 MPa. The stone and the mortar class are adopted to be as 50. The masonry design compressive strength is assumed to be of 4,4 MPa but the ultimate crushing strength is of 8,8 MPa; the Young's modulus is of

E=7000 MPa; the shear modulus is of G=3500 MPa but the Poisson's ratio is as $\nu=0$. The tensile strength is neglected.

In the masonry there are five openings and a balcony door opening (Fig. 1 a-d). Their vertical axes are almost identical. A single internal helical stone stairs (Fig. 1) ascends to the balcony level. One step is shown in Fig. 1 e. The short parts of the adjacent steps are overlapped entirely one above another and thus they form a central vertical cylindrical stone newel with a diameter of 16 cm. The steps are rigidly built into the masonry wall inner surface.

The principal damage of the minaret masonry is series of particular almost vertical cracks at the foot of the main prismatic part approximately between the +7,63 m and +10,18 m levels (Fig. 1a). The cracked sector is 2÷3 m high. In horizontal plan the cracks predominate around the opening axis; they occupy about 110 cm, i. e. 20÷25 % of the cross-section external perimeter and probably penetrate deep into the wall thickness. It is need to check the minaret safety.

3. CONCLUSIONS FROM FULL-SCALE DYNAMIC EXPERIMENTS

The full-scale dynamic tests have been done of the minaret part between the ±0,00 m and +25,41 m levels (Fig. 1) [3]. The transducers have been arranged at seven levels in the two principal examined directions - parallel (superscript '$\parallel$') and orthogonal (superscript '$\perp$') to the opening axes. The free vibrations have been initially generated by an nearly located oscillating equipment.

As a result of the experimental analysis the following facts and conclusions are derived [3]. The oscillated minaret part has been registered above the +9,05 m level. The lower part has been almost immovable. The average values of the basic natural periods in the parallel and orthogonal directions are as $T_1^{\parallel}=1,195$ s and $T_1^{\perp}=1,060$ s respectively. The structure is more flexible in the parallel direction. Probably this fact is due to the presence of the essential cracks predominant from the openings and passed over the masonry wall. The period value difference of about 13 % corresponds to a smaller horizontal stiffness of about 28-30 % in the parallel direction. The bending and shear strains are commensurable. The shear strains are stronger expressed in the first direction due to the available cracks. The effect of the soil settlements on the minaret dynamic characteristics is negligible. The minaret twisting vibrations can be also missed with an error of about 5÷7 %. The minaret seismic bearing capacity is strongly reduced due to the structural damages (first of all cracking). It is necessary urgent structural strengthening since in the present state the minaret can not sustain a future strong earthquake motion. A cantilever beam-column model resisting to bending and shear is fit for analysis.

4. ANALYSIS AND DESIGN OF THE MINARET STRUCTURE

The masonry part of the minaret structure between the levels of +5,57 m and +30,55 m is studied (Fig. 1). It is modeled as a vertical plane cantilever beam-column rigidly fixed at its bottom to the minaret low body (Fig. 2). In order to consider all structural

features the beam is divided into 20 finite elements. The basic (main) cross-section part
of the lower three elements is a rectangle with a central circular hole but the rest upper elements have ring cross-sections.

The characteristic wind pressure on the minaret vertical plan at a height of 10 m above the terrain level is given as 70 daN/cm^2 according to [4]. An seismic intensity of 7-th degree in the range MSK is adopted according to [5]. The second group soil is assumed. The structure seismic response coefficient R, the significance factor C and the seismic ratio K_c are as R=0,25, C=1,5 and K_c=0,1. The structure is computed under six loading combinations: dead loads, wind static and dynamic loads, a wind load (resonance, resonance static and resonance dynamic components), a horizontal seismic action, a vertical seismic incident, a horizontal seismic action plus 30 % unidirectional static and dynamic wind loads.

The linear elastic dynamic and static analyses and design of the minaret structure are performed by a FORTRAN IV special computer program ROKU [1] elaborated for complex study of high-rise tower-type structures by the FEM using a refined discrete plane beam-column model. The following factors are taken into account (Fig. 2): the horizontal displacement v_i, the rotation φ_i and the vertical displacement u_i at each node i; the bending, shear and axial strains and the corresponding elastic stiffnesses EI_i, GA_i and EA_i of each element i; uniformly distributed masses m_i and/or nodal concentrated masses M_i and their corresponding mass moments of inertia i_{mi} and I_{mi} and dynamic loads; the compressive normal force effect. The damping is neglected.

The linear dynamic, static and stability analyses of the model are accomplished by the FEM [1]. The well known uniform beam-column finite element of six DOF is used. The eigenvalue and eigenvector problem is solved by an available standard procedure using the Jacobi's method. The seismic loads, the dynamic and resonant wind loads are determined by the response spectrum method according to [4] and [5]. The modal responses are combined using the SRSS formula. For each specified loading combination the structure static stability is verified and then the higher-order bending moments caused by the vertical loads on the deformed beam axis in small displacements are computed by the iterative Vianello-Dischinger's method. The cross-section bearing capacity is checked by the stresses.

5. SOME RESULTS FROM THE ANALYSES AND DESIGN

The model is analyzed separately in the two principal directions. The first five natural periods and mode shapes are examined. The solution in the parallel direction is initially obtained by the elastic isotropic stiffnesses of all elements. In the next solutions in this direction the bending stiffnesses of the three lower cracked elements 3, 4 and 5 (Figs. 1, 2) are reduced gradually by a little step while the basic period value is obtained approximately equal to the experimental one ($T_1^{\parallel}$ =1,195 s). This solution was reached by almost 31,4 % reduced elastic isotropic bending stiffnesses of such elements and it is adopted as final. In this practical way the real elastic-plastic bending stiffnesses B_i of the cracked beam elements are approximately determinate.

By these stiffnesses many others solutions are still accomplished: neglecting shear; by the 31,4 % reduced shear and axial stiffnesses of the cracked elements, considering the first mode only, etc.

Some results from the dynamic analysis are given in Table 1. The following symbols are used: T_j (j=1, 2, 3, 4, 5) are free vibration periods; A_i and I_i are the cross-sectional area and the moment of inertia and D is a relative difference.

Table 1. Comparison of the examined periods of the minaret free vibrations

T_i ↓	By elastic stiffnesses			By 31 % reduced bending stiffness of the elements 3, 4 and 5 (B_i≈0,69EI_i)						*Test analysis* [4]			
	$T_i^{\perp}$ (s)	$T_i^{\parallel}$ (s)	*D* *(%)*	$T_i^{\parallel}$ (s)	*D* *(%)*	$T_i^{\parallel}$ (s) G=0	*D* *(%)*	$T_i^{\parallel}$ (s) 0,7EA_i	*D* *(%)*	$T_i^{\perp}$ (s)	*D* *%*	$T_i^{\parallel}$ (s)	*D* *%*
T_1	1,11	1,13	*1,6*	1,20	*5,9*	1,19	*0,4*	1,19	*0,0*	1,06	*4,7*	1,20	*5,6*
T_2	0,18	0,19	*1,5*	0,19	*3,0*	0,19	*2,1*	0,19	*0,4*	-	-	-	-
T_3	0,08	0,08	*1,6*	0,08	*1,8*	0,08	*4,3*	0,08	*0,8*	-	-	-	-
T_4	0,06	0,06	*0,0*	0,06	*0,0*	0,06	*0,0*	0,06	*6,2*	-	-	-	-
T_5	0,05	0,05	*1,5*	0,05	*0,9*	0,04	*5,8*	0,05	*1,5*	-	-	-	-

Some results from the static analysis in the parallel direction are included in Table 2. The solutions are accomplished by the reduced bending stiffnesses of the cracked elements. The symbols N_i, M_i and Q_i means respectively a normal force, a bending moment and a shear force at the lower end of the beam element i and M_i^h is a higher-order bending moment as an additional external load at a node i. The sign conventions are shown in Fig. 2. The quantity absolute values are given. The forces are in kN, the moments are in kNm and the displacements are in cm.

Table 2. Internal forces, higher-order moments and displacements in some typical nodes

Loads →	Under wind loading combination						Under seismic action combination				
	Elastic solution	Solutions by 31% reduced stiffnesses of the cracked elements 3, 4 and 5					Elastic solution	Solutions by 31% reduced stiffnesses of the elements 3,4,5			
Quantities ↓	due to wind load	Due to wind load	Static wind load	Dynamic wind	Without shear	First mode only	by horizontal motion	Horizontal motion	Without shear	First mode only	Vertical motion
$N_1^{\parallel}$	892	892	893	894	892	893	813	813	813	813	857
$M_1^{\parallel}$	1934	1993	1337	656	1990	1991	222	205	205	196	-
$Q_1^{\parallel}$	101	101	73	28	101	100	18	17	17	14	-
$N_4^{\parallel}$	604	604	605	606	604	605	552	552	552	552	595
$M_4^{\parallel}$	1536	1575	1048	527	1573	1575	190	177	177	173	-
$Q_4^{\parallel}$	87	88	59	28	88	87	17	17	16	14	-
$v_{21}^{\parallel}$	9,8	11,1	7,4	3,7	11,1	11,1	2,2	2,4	2,3	2,4	-
$M_1^{h,\parallel}$	33	38	25	13	37	38	5	6	6	6	-

$M_4^{h,\parallel}$	32	37	25	12	37	37	5	6	5	5	-

Some diagrams of the design internal forces and the horizontal displacement diagram caused by some loading combinations in the parallel direction static analysis are plotted in Fig. 3.

6. DISCUSSION OF RESULTS AND EFFECTS OF SOME FACTORS

The change of the real 16-gonal cross-section of the minaret masonry above the +8,65 m level (Figs. 1, 2) with its circumscribed ring section increase slightly the cross-sectional area and the axial moment of inertia with 2,6 % and 5,3 % respectively. The steps and the central stone newel increase also these characteristics up to 14,0 % and up to 9,3 % but the masonry openings decrease such ones up to 11,8 % and up to 23,5 % respectively. The strongly expressed difference in the elastic bending stiffnesses of each of the elements 1, 2 and 3 in the two examined directions ($I_i^{\parallel} \approx (0,61 \div 0,83) I_i^{\perp}$, i=1, 2, 3) as well the presence of five openings in the studied minaret part ($I_i^{\parallel} \approx (0,93 \div 0,99) I_i^{\perp}$, i=5, 8, 11, 14, 17-19) and available asymmetrical cracks in elements 3, 4 and 5 disturb the minaret axial or regular symmetry but a vertical central plane of symmetry including the opening axes is available. The structure is more flexible in the parallel direction.

The values of the free vibration fundamental periods $T_1^{\parallel}$ and $T_1^{\perp}$ obtained by the elastic solutions differ from the experimentally registered in [3] in about of 5 % only. The corresponding periods $T_j^{\parallel}$ and $T_j^{\perp}$ (j=1, 2, 3, 5) differ in between about of 1,6 % only. The periods in the parallel direction are a little larger ($T_j^{\parallel} \geq T_j^{\perp}$). The fourth mode shape is connected with the vertical vibrations.

The reliable results are in the parallel direction since the internal forces, displacements and stresses are larger than the corresponding ones in the orthogonal direction in the elastic solution too. This difference increases if the elastic bending stiffnesses of the cracked elements 3, 4 and 5 in the first direction are diminished by 31,4 % to reach the period experimental value $T_1^{\parallel}$ =1,195 s. Then the internal forces and displacements due to the wind load are a little extended as the quantities $M_1^{\parallel}$, $Q_1^{\parallel}$, $v_{21}^{\parallel}$ and $M_4^{h,\parallel}$ are enhanced by 3,1 %, 0,6 %, 13,4 % and 2,4 % respectively but the same magnitudes caused by the seismic action are reduced by 7,9 %, 3,8 %, 7,3 % and 3,1 %.

The shear strain effect is insignificant. The reduction of the cracked beam shear stiffness by still 31,4 % changes periods with of 0,1÷1,5 % only. Neglecting shear the periods $T_j^{\parallel}$ decrease by 0,4÷5,6 % only; the internal forces and displacements diminish by up to 0,2 % and 0,5 % respectively. The structure is subjected exclusively to bending because of its slenderness. The axial stiffness reduction by the same 31,4 % increases the period T_4 by 6,2 %.

The most critical section is the horizontal one through the node 4 (at the foot of the minaret main high prismatic part) (Fig. 3) since the largest normal stresses due to the reliable wind loading combination (including dead loads and wind static and dynamic loads) are obtained at it by the elastic stiffnesses and the Navier's formula. They are as $\sigma_1 \approx 4{,}95$ MPa (compression) and $\sigma_2 \approx 3{,}91$ MPa (tension) and exceed the corresponding stone masonry design strengths. The corresponding stresses due to the dead and wind static loads are as $\sigma_1 \approx 3{,}99$ MPa and $\sigma_2 \approx 2{,}95$ MPa but under dead and seismic loads they are as $\sigma_1 \approx 1{,}10$ MPa and $\sigma_2 \approx 0{,}16$ MPa. The seismic compressive stresses at the section 4 can be withstood but even under dead and wind static loads they can not be sustained. The bending stresses are essential. The wind compressive axial stresses σ_N are only about of 11,7 % from the bending ones σ_{1M}, but under the seismic loads this ratio is of 75,3 %. The maximal total shear stress in this section due to the same wind loading combination is determined as $\tau_{x,max} \approx 0{,}15$ MPa only. Even without tension exclusion the wind compressive stresses in the lower 33 % portion of the high prismatic part exceed the stone masonry design compressive strength too.

The reliable wind loading combination causes considerably larger internal forces (as 5÷10 times) and displacements (as 4÷5 times) in the structure than the seismic loading combination. The loading combination covering dead, seismic and 30 % wind loads is not reliable also. For this structure the seismic action effect is small because of the lower design seismic grade at the site and the minaret small self weight (about 814 kN total but only 546 kN above the node 4).

The resultant of the wind static load will be reduced by 89,8 % if a refined spatial shell or 3D structural model of the minaret structure is used. For the beam-column model the distributed resultant characteristic values of this load at levels of 10 m, 20 m and 40 m computed on the minaret vertical plan are as q_{10m}=119 daN/m, q_{20m}=148,8 daN/m and q_{40m}=184,5 daN/m. For the shell or 3D models the corresponding resultant values, calculated by an numerical integration in circumcircle using the specified in [4] static wind load in circumscribed circle at intervals of 15^0 and applying the Simpson's rule, are as q_{10m}=62,7 daN/m, q_{20m}=78,4 daN/m and q_{40m}=97,2 daN/m. If the same load is interpolated in circumcircle by the first five harmonic load components considered of the shell or 3D models, the resultant values obtained by an analytical integration in circumscribed circle are as q_{10m}=61,5 daN/m, q_{20m}=76,9 daN/m and q_{40m}=95,4 daN/m. The last values are smaller by only 1,9 % than the corresponding ones of the same models but for the standard load.

The effect of the wind dynamic load on the internal forces in the section 4 and on the top horizontal displacement v_{21} reaches to about 50 % towards the wind static load contribution. The dynamic load basic effect is caused by the first mode shape. Its contribution into the internal forces in the section 4 and the top horizontal displacement comprises about 99 % from the whole effect due to the first five mode shapes examined. The first mode shape effect caused by the seismic action is of 98,1 % to the bending moment and of 82,3 % to the shear force in this section. According to [4] the wind dynamic load should be considered for the first mode shape at least. The resonance wind load is unnecessary for this structure keeping [4] also.

The horizontal displacements of the minaret upper part due to the characteristic wind load are obtained too large by the reduced bending stiffnesses of the cracked elements. At the masonry top and at the balcony level they are as v_{21}=11,1 cm and

v_{17}=7,4 cm and the displacement-to-rise ratios are about of 1/180 and of 1/271 respectively. The column shortening is of 1 mm only.

The structure safety against static buckling is not large. The safety factors against loss of stability due to the wind loading combination are determined by the elastic stiffnesses as $\gamma_1^{\parallel} = 7{,}99$ and $\gamma_1^{\perp} = 8{,}28$, but by the reduced bending stiffnesses of the cracked elements the safety factors against wind and seismic buckling are as 7,07 and as 7,76 respectively. The higher-order bending moments are very small (up to 2,4 %) in comparison with the basic ones and can be neglected due to the minaret small weight.

The minaret analysis under vertical or inclined seismic motions to the horizon as well the vertical vibration study are useless for this model. The vertical seismic forces increase or decrease the internal compressive normal forces up to about of 7,8 %.

The elements and nodes with big masses get considerable dynamic loads. In the thickened element 16 arise seismic distributed loads which are larger by 4,4÷7,8 times than these ones in the adjacent element 15 (Figs. 1, 2), which has of 3,8 times smaller distributed weight and of 7,60 times smaller weight moment of inertia. The wind dynamic load on the element 16 is of 2,2 times larger than the wind static load, while on the element 15 the similar dynamic load is of 35,8 % smaller.

7. PRINCIPAL CONCLUSIONS AND RECOMMENDATIONS

If an axial symmetry is absent, several (at least 2-3) independent directions should be examined using the plane beam-column model. Reduced isotropic bending stiffnesses of the cracked beam elements are appropriate for analysis in the direction with predominant cracked zones. Usually the analysis in the direction in which the structure is most flexible is reliable.

The wind static load should be more correctly determinate by spatial shell or 3D (solid) models keeping the design code [4]. This will considerably reduce the wind dynamic load also. As a result of this the structure under the wind loads will essentially relieve. Moreover spatial models as beam-column, shell and 3D idealizations and general-purpose computer programs elaborated on the basis of the FEM can be used. Then orthotropic stiffnesses of the cracked elements or special finite elements (gap, hook) should be used in the cracked zones. Two orthogonal horizontal seismic actions and an eventual vertical action can be simultaneously assumed.

Checks on the safety of the monument can be carried out for both elastic and plastic behaviour of the cross-sections although the response spectrum method is strictly valid for linear elastic analysis only. The structure ultimate bearing capacity check can be accomplished proving the compressive stress maximal absolute values calculated by the Navier's formula at the critical sections after tension elimination. This check can be facilitated if the ultimate bearing capacity area is preliminary plotted by the limiting curve of the relation "bending moments versus normal forces (M-N)". The normal stresses ought to be computed by the elastic cross-section. The lower portion of the high main prismatic body of the original (unstrengthened) structure is unsafe.

The obtained numerical results verify the basic available experimental results exception the assumed large shear strain effect and the reliable seismic action. The results get from the dynamic and static analyses are also similar to these ones obtained from analysis and design of the particular minaret in Eger (Hungary) [2]. The principle difference is that the seismic loads have been reliable due to the minaret large self weight and bending stiffnesses as well to the consideration of the lower thickened part and the foundation. The wind analysis is not presented in [2].

Nevertheless the minaret structure has stood for several centuries with small visible damages (cracking) only. Probable reasons for this are: some masonry tensile strength; a masonry higher ultimate compressive strength than the design value; overestimation of the design loadings (especially wind static and dynamic loads on the beam-column model) in relation to real ones.

An urgent strengthening of the stone masonry is need to be made only from inside by an appropriate way to restore the minaret normal safety in accordance with the current Bulgarian design codes. The strengthening should cover in height the whole internal surface of the masonry wall or at least the unsafe lower portion of the main prismatic part. Two alternative ways for repair and strengthening are discussed in [2]: prestressing by vertical tendons and construction of an additional internal reinforced concrete skin. The second way has been adopted as more rational and it has been applied. The numerical reanalysis and redesign as well the full-scale dynamic and static experiments of particular phases during the minaret repair are necessary.

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A PROCEDURE FOR EVALUATING THE SEISMIC VULNERABILITY OF HISTORIC BUILDINGS AT URBAN SCALE BASED ON MECHANICAL PARAMETERS.

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ABSTRACT

The paper presents a procedure aimed at the evaluation of structural damage in masonry historic buildings subjected to earthquakes. The approach chosen is based on a failure analysis of the structures through the identification of feasible collapse mechanisms and the calculation of their associated failure load factors. The mechanisms are ranked in terms of their likelihood to occur depending on external constraint and quality of materials and the results manipulated to produce a measure of vulnerability. The methodology, TOSQA99, has been recently further developed to include a wider range of possible collapse mechanisms and an accurate modelling of the continuity among orthogonal walls. The capability of the procedure is discussed by an application to a group of buildings surveyed in the historic centre of Nocera Umbra, stricken by the 1997 Umbria-Marche earthquake, Italy.

1. INTRODUCTION

The programme TOSQA99, developed over the past five years [1,2], is based on a limit state, lower bound, mechanical approach analysis of the individual bearing walls forming a masonry building. The analysis is static equivalent and aims to predict the level of ground acceleration, which will trigger the onset of a specific failure mechanism. On this basis, it is possible to produce a projection of damage modes and levels of vulnerability for individuals or groups of buildings, in relation to expected levels of peak ground acceleration at the site. It is also possible to analyse the reduction in vulnerability obtainable by introducing selected types of strengthening.

Unlike other statistical procedures [3,4,5,6], this methodology can be applied to medium sized samples of buildings without renouncing a proper and sufficiently detailed analysis of the geometrical, typological and structural parameters which qualify the analysed buildings. This specific feature of TOSQA99 is strictly related to the way in which the data collection is organised: the on site inspection concentrate on those parameters which can directly qualify the seismic performance of masonry buildings and can be surveyed from the street.

In order to optimise the survey time, the required data are divided into sub-groups, by organizing the investigation in two steps. In the first, the operator is required to recognise, within the sample the recurring characteristics in terms of structural typologies, masonry fabrics, quality of materials, and these parameters are then quantified for the identified types. This set of data usually directly relates to the local construction tradition and availability of materials. The second step consists of a collection of specific information for each building, such as height, length, and thickness of each inspectable façade, number of storeys, strengthening devices and so on). In this phase, the operator is also required to associate to each building, a structural typology and masonry type from those surveyed in the previous phase. This association, represents a fundamental step in the process of investigation of a building, because it enables to associate to each external wall of a building a set of information regarding the interior, which otherwise would be unknown. However, this association in itself necessarily implies a margin of uncertainty, and this is also recorded on the survey form.

On the basis of the information collected, the procedure then associates to each external wall the loading and constraint conditions deduced from the survey and identifies a group of feasible mechanisms, for each one of which the ultimate load factor is calculated. This is expressed by the index *ESC*, equivalent shear capacity, in terms of percentage of gravity acceleration. The mechanisms are then ranked in terms of their associated *ESC* and the extension of the building involved in it, and a judgement is then made of the danger they pose. This represents the vulnerability of the building.

The procedure is developed to work directly within the database, so using a worksheet package it is possible to input directly the data in a electronic form, which is automatically stored into the database sheet, and this is directly accessed via a number of macro to calculate the failure load factors.

In the mechanical formulation, a central role is played by the mechanical quality of the masonry. An accurate study of the mechanical behaviour of the masonry has been carried out in order to identify those variables, which can influence the seismic performance of a wall. This masonry modelling has enabled to quantify the frictional forces displayed along orthogonal wall connection.

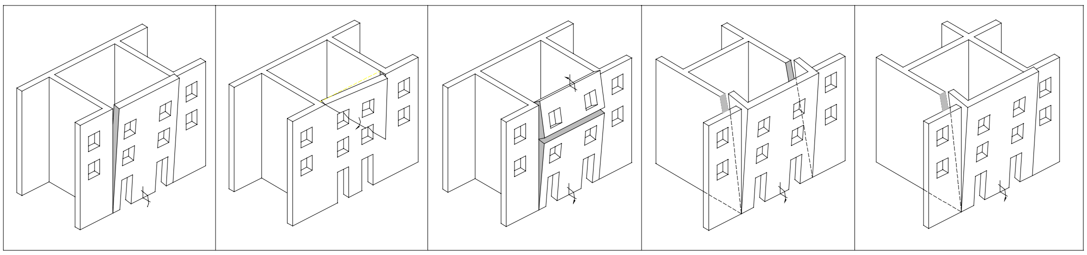
Although the mechanical modelling has been so far developed for very regular kinds of masonry, it can also be applied to more chaotic patterns, on the basis of appropriate assumptions.

2. FAILURE MECHANISMS DESCRIPTION

The formulation proposed in this application takes into account five common kinds of out of plane collapses, as they have been previously identified by post earthquake damage inspections [7, 8], and schematically sketched in Figure1:

- A1 – Façade overturning, with hinge at the base;
- A2 – Asymmetrical façade overturning identified by an oblique hinge;
- A4 – Vertical arch effect.
- C1 – Façade overturning with two orthogonal wings;
- C5 – Façade overturning with one orthogonal wing;

Each of the above mechanisms can develop either over the whole height of the façade or involve only some of the storeys. To make an example, mechanism A1 can occur only for the upper storey, or for the two upper ones or for the whole wall.



(A_1) (A_2) (A_4) (C_1) (C_5)

Figure.1 – Out of plane mechanisms taken into account in the analysis

Of the analysed mechanisms, mechanisms A1 and A2 refer to the overturning of the façade without the participation in the mechanism of the side walls. The mechanisms are however influenced by the level of connection with those walls which operate a restraining action, along the vertical edges of the façade. A1 assumes that the wall forms horizontal hinges, in turn at each floor level. Mechanism A2 is associated with only a portion of the façade overturning around a diagonal hinge. Mechanisms C1 and C5 are extensions of mechanism A1 when respectively both or one side walls participate to the overturning [9].

The essential parameters influencing the final value of shear capacity are the relative dimensions of the units and the level of connection between orthogonal walls. This will determine not only whether and to which extent portions of the side wall will participate in the mechanisms, but which failure pattern may occur.

The level of connection of the façade implies the calculation of the restraining action acting along the quoins. As it depends on the masonry characteristics surveyed, the model implemented for the in-plane shear assumes frictional behaviour among courses of blocks and it is described by:

- the average height of the horizontal layers;
- the staggering of blocks of superposed courses;
- the specific weight of the blockwork
- the friction coefficient of the contact surfaces.

According to this model, for equal values of the friction and specific weight of the material and dimension of the masonry panel analysed, blockwork characterised by small height/staggering ratios shows higher in-plane strength than blockwork characterised by higher ratios. Also this ratio and the geometric ratio of the panel will define the critical angle along which the wall will develop a diagonal crack and whether the failure in plane mechanism will be characterised by overturning or sliding motion. This will also be a function of the friction coefficient chosen. As it is obviously not feasible to measure friction coefficients associated with every type of masonry surveyed on site, this association is performed by use of values which have been identified by experimental work carried out by others on similar masonry types [10]. Also a parametric study by the present authors is reported elsewhere [11].

So depending on the geometry of the blockwork and on the quality of the connection between orthogonal walls as surveyed on site, for each façade a specific value of the restraining action developed by the connection and of the portion of party walls participating in the overturning mechanism is calculated. The restraining action developed along the quoins through friction is then composed with the vertical loads directly acting onto the façade and the horizontal equivalent seismic action, to define the limit equilibrium conditions.

Two other parameters are crucial in the definition of the mechanisms and in the associated value of ESC: the presence of ties and the restraint operated by the horizontal structures. The values associated with each of these parameters are chosen case by case depending on the information obtained through the survey.

The presence/absence of strengthening devices is a crucial parameter, which yields a first selection of possible collapses. For instance, if no ties or ring beams are recorded for a given façade, the mechanism A4 will be excluded. On the contrary, if a ring beam is registered at the top of the building, the A4 will occur while neither the A1 nor the C1 could develop.

When the surveyed level of connection is good, the *ESC* calculated for failures types A1 and A2 is usually so high to make their triggering very improbable. Indeed the results show that for good connection a crack will form for lower values of collapse load factors in the party walls and a portion of these will overturn with the façade as sketched in cases C1 and C5. The occurrence of either of these two will depend on the relative level of connection at the two edges of the façade. This result is confirmed by post earthquake damage surveys and experimental evidence (reference), as usually mechanisms of types A1 or A2 are associated to very weak edge connections, while the opposite is true for mechanisms of type C1 or C5.

Finally, a further possible mechanism considered is the development of diagonal cracks due to acceleration in the plane of the façade. The model adopted is the same described above for the blockwork, and depending on the relative values of the parameters, the crack will develop for a given angle to isolate either a triangular or trapezoidal upper portion which will either overturn or slide over the lower one.

3. APPLICATION TO THE CASE STUDY OF NOCERA UMBRA

3.1 Nocera Umbra: buildings typology and structural characteristics

Nocera Umbra is located in central Italy, on a narrow and elongated hilltop close to the Appenines and Assisi. The historic centre, enclosed by city walls and composed of around 220 buildings, is typical of a medieval town, with buildings arranged in long parallel and almost concentric arrays. Latter interventions, mainly due to historic earthquakes, occurred in 17th and 18th centuries, have modified to some extent the original urban fabric. Most of the more recent growth of the town as however occurred beyond the city walls, so that the original nucleus is relatively well preserved, with a number of buildings of monumental value. The buildings are arranged in long terraced arrays, with common party walls and variable number of storeys on the hill side (up to 2 or 3) and valley side (usually 4 or 5, with a maximum of 6). The typical house is usually formed by one or two masonry cells, depending on the depth of the block, with a staircase running, usually but not necessarily, along the party walls.. In the following a more accurate description of a few representative units is provided.

3.2 Characteristics of the surveyed buildings.

In a recent on site inspection of the historic centre of Nocera [11] a detailed survey was carried out of some buildings, chosen as representative examples both of the types of masonry fabric and of the internal layout and horizontal structures. Of the eight buildings surveyed, a group of three (cadastral reference numbers *394*-395-396) (Fig.2) forms a small block shaped as a wedge at the eastern edge of the hill with 3 to 4 storeys on the valley side and 2 to 3 storeys on the hill side. The depth of the block is formed by one single masonry cell of rather constant width and depth variable from 4 to 6 m. The level of alteration can be easily read from the pattern and layout of openings, however it conserves original floors with the traditional timber joists and tiles. Number 394 at the left end of the block suffered the collapse of the upper portion of the end wall and the partial collapse of the two orthogonal walls connected to this. The other two units, which presented some traditional ties at the upper levels, suffered vertical cracks and other minor damage. Two leaves of rubble stones with a sandy core form the masonry of unit 394. The stones are of variable dimensions from small 100x100 mm pebbles to 150x300 units (Fig. 3a). The beds are reasonably regular with good staggering and lime mortar bed joints varying from 10 to 15 mm. The other units have the most common type of masonry in Nocera, formed by very regular roughly dressed stones of typical dimensions 100*250 mm, Fig 3b. The quality of connection between orthogonal walls is shown in Fig. 4.

3.3 Analysis of units 394 395 396

For the unit 396 walls no. 12-1, 12-2 and 13 (fig. 5) have been analysed as they show the highest slenderness. Walls 12-2 and 13 have the same height and thickness but different loading condition. For these 3 walls the height of the stone block has been set to 150 mm, while the overlapping length is 80 mm to account for the irregular shape of the bedding surface. The coefficient of friction is taken to 0.3,which is a relatively modest value for unpolished stone surfaces.

For each of these walls the 3 cases of poor and good connection on both edges and good connection on one edge only have been considered. It is also assumed that the level of maintenance of the roof and floors is good, so the restraining action exerted by those is accounted for the maximum developed in relation to the typology. In the following results are discussed in details for wall 12b.In the graphs of fig.6 the slenderness refers to the portion of wall which is actually mobilized by the mechanism.

In the case of good connection at both edges of the wall the lowest load factor is represented by the in plane sliding mechanism, triggered when the external acceleration reaches the same value as the friction coefficient. The out of plane overturning of the upper floors of the façade also has a load factor close to 0.30, as indeed the mechanism of in plane overturning for the whole façade. All other load factors are closer to 0.40, with mechanisms C1 and C5 slightly lower than A1. The minimum values are associated with the overturning of the upper two storeys.

In presence of good connection with only one of the sidewalls the mechanism of partial overturning of a portion of the façade A2, results to have lower load factor than the overturning of the entire wall, except than for the upper floor. Also in this case the minimum load factor for mechanisms A1, C1, and C5 is obtained in association with the mobilizing of only the two upper floors of the structure. Mechanism C5 shows lower values of C1, as it should be expected by the asymmetrical connection.

Fig.2 Survey of the block units 394,395 and 396. Construction layout and damage pattern.

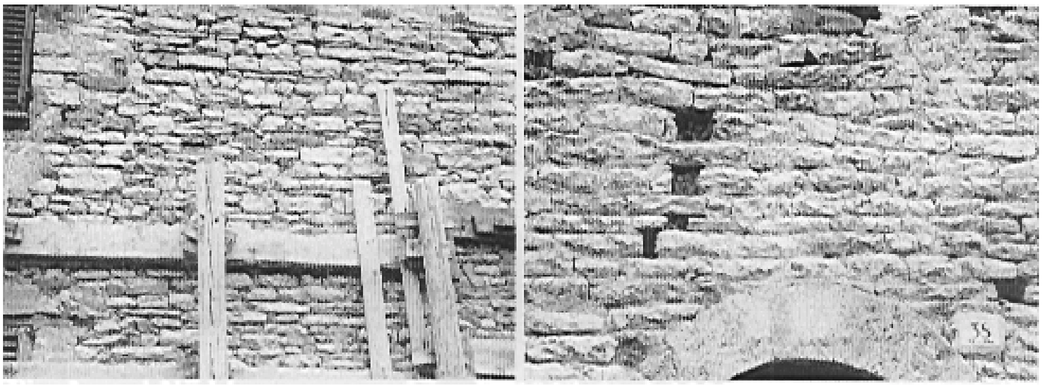


Fig. 3a and 3b: Masonry types for units 394(left) and 395 (right)

Fig. 4 Unit 396 showing good connection.

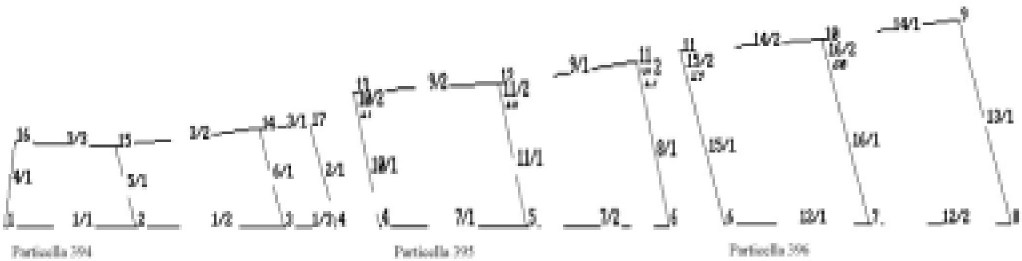


Fig. 5 Subdivision in cadastral units and key to wall numbers

In the case of poor connection, obviously the minimum load factor is associated with the complete overturning of the whole façade (A1), which has a value slightly greater than 0.1 a/g. The two mechanisms with participation of the sidewalls show obviously values just above the friction threshold. It is however worth noticing the crucial role of the presence of the ties. In fact while in the previous two cases the load factor associated with A4 were well above the range of interest, here the presence and efficiency of the ties, might represent the difference between collapse of the façade or presence of horizontal cracking. It also means that the structure can withstand accelerations twice as high.
In conclusion the possible behaviours of wall 12b are summarized in table 1, depending on the level of connection and the associated minimum load factor.

Table 1.

Connection with lateral walls	**Good on both sides**	**Good on one side**		**Poor**	
Other connections			Ties		Ties
Minimum load factor	0.30	0.16	0.22	0.11	0.22
Associated mechanism	In plane sliding or overturning	A2		A1	A4
Portion triggered	Upper triangular part of the façade	Triangular portion of whole façade or upper storeys		Entire facade	Upper portion

If the choice of the most likely mechanism were based solely on the value of the minimum load factor, the most likely condition would be the occurrence of A1 and the vulnerability of this wall is high. However the presence of ties has been recorded and it is sufficiently regular to assume it to be efficient. Therefore the most likely limit value for this façade is *ESC* = 0.22 associated with mechanism A4 which yields a medium level of vulnerability, and would guaranty the capacity of the wall to withstand an earthquake of second category.

The efficiency of the connection to the sidewalls, of more difficult ascertainment, can be guided in first instance by the regularity and quality of the masonry fabric of the wall and the presence of identified discontinuity (pre-existing cracks or outward leaning of the wall), and secondly by the position of the building within the block and of the wall under consideration within the building. For instance, for a building at the end of the block, such as unit 396, with three free walls with an homogeneous fabric and dimensions of the block that correlate well with the overall thickness of the wall, it can be expected that the end wall has a good connection with the two sidewalls and that those might or might not have a good connection with the adjacent and orthogonal walls at the other edge.

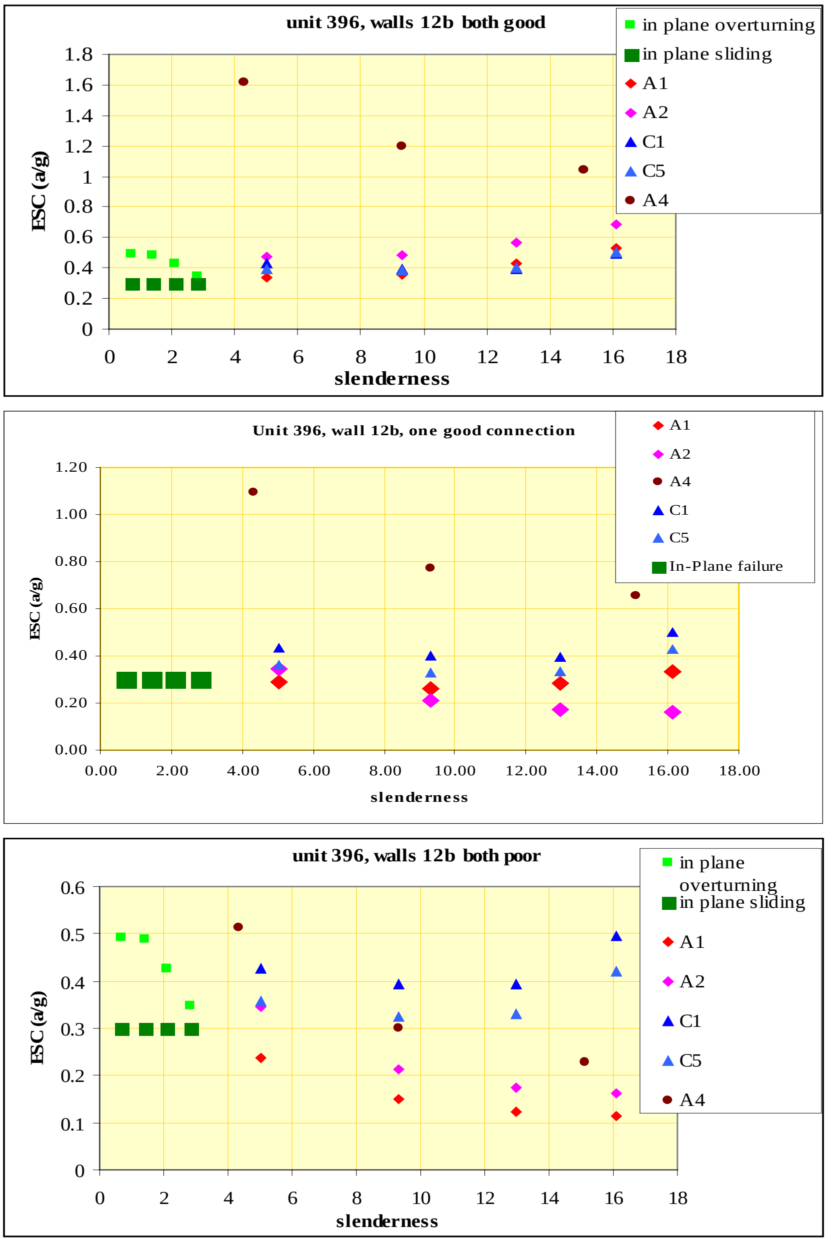


Fig. 6 Distribution of collapse load factor for various condition of connection

So being conservative it can be assumed that wall 12b has only one good connection at the end corner edge, in which case if the ties are also active, the global A2 will be prevented by the presence of them at the inner edge, in favour of A2 of the upper two storeys associated with a load factor 0.22. So either this last mechanism or the A4 associated with poor connection will occur. This is the type of screening of the results that the procedure performs in order to achieve the final judgement on vulnerability.

In the case of post earthquake survey damage the identification of a given mechanism is further checked with the recorded crack and damage pattern to evaluate whether they are correlated or not. For unit 394 the walls 1-1, 3-2 and 4 were analysed, to reproduce the failure observed. The results show that the minimum value of the load factor is obtained for the mechanism C1 of wall 4 ESC= 0.16, developing on the upper 2 floors and this is matched by the value of Esc=0.15 for wall 1-1 for mechanism A2 and C5, reproducing the failure shown in fig.2.

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